GEOTECHNICAL APPENDIX
UPPER YORK CREEK ECOSYSTEM RESTORATION PROJECT
ST. HELENA, CALIFORNIA

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1.0 INTRODUCTION

This document presents the findings of geotechnical studies on an existing Upper York Creek earthen dam located along York Creek approximately two miles northwest of the City of Saint Helena in Napa County, California. The primary objective of the studies is to evaluate the subsurface conditions of the dam's foundation and the nature of the embankment fill as they relate to the various options for removing the dam and the sediments that accumulated upstream.

The dam is considered an impediment to the upstream passage of anadromous fish species, in turn degrading fish spawning and rearing habitat. The California Department of Water Resources (DWR) reported that in the past the dam has experienced overtopping failures releasing silt downstream.

2.0 EXPLORATIONS

Figure 1 summarizes all the exploration locations conducted over the past 13 years (Blackburn Consulting, 2005). Copies of the boring logs from these subsurface investigations are found in **Appendix G1**. The City of Helena through its consultant Blackburn Consulting conducted the most recent investigation in October 2005, which consisted of 2 borings through the crest of the dam and below the original channel. These borings were supplemented with 4 piezometers located near the dam abutments and 2 within the landslide east of Spring Mountain Road (**Appendix G2**). Three inclinometers were installed, one located next to the piezometer at the left abutment and 2 next to the piezometers within the landslide area. Five test pits were performed along the upstream of the dam and within the existing York Creek Channel. Finally, 6 ground penetrating radar (GPR) was conducted by Earth Imaging Geologic Services within the channel upstream of the dam.

In October 2003 the U.S. Army Corps of Engineers contracted Treadwell & Rollo to perform 5 borings with 2 borings located along Spring Mountain Road, one in the spillway chute and 2 along the western edge of the spillway. These borings ranged in depth from 15 feet in the spillway to 36.5 feet in the left abutment. This exploration is limited to portions of Spring Mountain Road and left abutment of the dam.

In July 1993, Huntingdon conducted 6 borings along the crest of the dam varying from 10 feet to 21 feet below the existing ground surface.

3.0 GEOTECHNICAL AND GEOLOGIC CONDITIONS

3.1 GENERAL GEOLOGY

The Upper York Creek site lies within the Coastal Ranges geomorphic province of California. The Coastal Range province is characterized by a series of nearly parallel mountain ranges and alluviated valleys that trend obliquely to the coastline in a northwesterly direction. The geologic units are composed of a heterogeneous mixture of intrusive, extrusive, metamorphic and sedimentary rock types, which exhibit varying degrees of tectonic deformation.

Based on the "Preliminary Geologic Map of Eastern Sonoma County and Western Napa County", miscellaneous field studies map MF-483, the site is in an area that is geologically complex. Geologic formations in the immediate vicinity of the site are as follows from oldest to youngest. Oldest is the Franciscan complex (Kjfs) rock of Jurassic to Cretaceous in age consisting of sheared shale, highly broken and sheared sandstone, and perhaps some meta-volcanics (greenstone), intruded by highly sheared serpentinite. Overlying the Franciscan complex are the Sonoma volcanics, Pliocene in age (2 million to 5 million yeas ago) consisting of pumicitic ash-flow tuff (Tst), locally welded or partially welded with intercalated bedded agglomeritic tuff, and perlitic ryholite (Tsrp) and a few relatively thin bands of bedded sedimentary deposits (Tss). At the base of York Creek canyon and underlying St. Helena are Quaternary older alluvial fan deposits less than 2 million in age.

Appendix G3 shows a part of the referenced preliminary geologic map.

3.2 SITE GEOLOGY

Figure 2 shows the cross section drawn along in a transverse direction from Spring Mountain Road to the crest of the dam. Subsurface conditions within the dam embankment consisted of fill overlying serpentinite bedrock. Natural undisturbed slope of the bedrock on the right, as looking downstream, abutment above the dam is estimated at about 1.4 H to 1V slope (horizontal to vertical) and steeper in other areas. The steepness of this slope is likely attributed to the presences of Franciscan sandstone, mapped as Kjfs in MF-483; although not investigated for this project, as the slope is steeper than the sheared Franciscan shale would likely support.

The section along the axis of the dam encountered serpentinite as foundation bedrock under the dam. Boring B-1(05) which was drilled closest to the right abutment encountered 15 feet of fill over serpentinite which extended to the bottom of Boring B1(05) to a depth of nearly 45 feet. The vertical extent of the serpentinite at this location is unknown. This boring appears to confirm that the contact between the intruded serpentinite and the Franciscan complex is either very steep to dipping into the right abutment, and according to the mapped trace trends nearly east-west. The contact of the serpentinite with the Sonoma volcanics is also very steep, nearly vertical, and trends in a northerly direction. The contact is exposed in the head scrap of the mapped landslide in the upper left abutment of the dam; the landslide occurred in the weaker, more highly weathered, near surface serpentinite material.

As the serpentinite is believed to be a relatively cold plastic intrusion, no indication of a baked contact or any contact metamorphism with the Sonoma volcanics, the intrusion likely occurred after the Sonoma volcanics had been laid down, probably during the mid-Pleistocene uplift and deformation that produced the present day Coast Range Mountains. According to MF-483 the mapped bedding within the Tsrp and Tss units generally dip very steeply to the east and locally adjacent to the Kjfs contact the bedding appears to have been overturned. The serpentinite intrusion thus occurred along the apparent intersection of two planes of weakness or faults. The intersection of the two faults may lie within the stream channel immediately upstream of the dam. If so, the highly sheared and weak nature of the serpentinite makes the material highly susceptible to stream erosion and scour, and, the fault intersection an obvious place for an erosion nick-point to begin. The faults themselves appear to be short and not associated with any active faults.

3.3 SITE GEOTECHNICAL PROPERTIES

The upper portion of the fill overlying the bedrock is composed of sandy silty, silty sand, and clayey sand mixed with gravel and cobbles. The gravels and cobbles are light gray and pink welded volcanic tuff and dark green and black serpentinite. At the highest point, the crest of the dam is at elevation +620 feet NGVD. The maximum thickness of the fill that forms the dam is 43 feet. The existing dam has a crest width that varies between 11 feet and 27 feet, and side slopes that are approximately 2.2H:1.0V upstream and 1.6H:1V on the downstream slope. The upstream and downstream face of the dam appears stable in areas that have not been affected by overtopping event.

Below the dam fill is serpentinite bedrock. The serpentinite bedrock is also green, black, and dark-reddish brown, friable, deeply weathered and intensely sheared. The serpentinite is described as an intensely weathered rock that has significantly weakened. The next stage of rock weathering is decomposed rock, which is resembling a soil. Based on laboratory testing results, the bedrock strength characteristics vary significantly with unconfined compressive strengths from 1,100 psf to 11,100 psf. The strength of the serpentinite is estimated based on the values of cohesion and frictional angles as measured from the direct shear tests *ASTM D 5607 Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force*. For the direct shear test data, the strength of the serpentinite is estimated to be 1000 psf cohesion and 37^0 friction angle.

The strength of the serpentine used in the design calculations should be viewed with extreme caution as core recovery during sampling was extremely poor, possibly indicating zones of material weaker than tested. It should be noted that the standard laboratory strength testing on samples of intact rock may not necessarily reflect the in-situ rock mass strength and deformation characteristics, especially for the serpentinite bedrock whose in-situ conditions are found to be friable, deeply weathered, and intensely sheared (Treadwell & Rollo, 2004). For example, based on the two boreholes adjacent to the proposed excavation face of the dam, the rock quality as indicated by the RQDs of zero is considered ‘very poor’, i.e. rock with numerous highly weathered joints spaced < 2 inches apart (FHWA design manual, undated). However, the laboratory tests resulted to relatively higher strength values, as high as 11,000 pounds per square foot (psf) for the unconfined compressive strength and about shear strength of 6,000 psf

calculated from apparent cohesion of 2417 psf, friction angle of 40.4 degrees and normal stress of 4500 psf (Borehole B-4).

Two failure planes labeled as recent slide ("active") and old slide ("dormant") are shown in cross section A-A of Figure 2. These interpreted landslides can be found in the January 16, 2006 Geotechnical Data Report prepared by Blackburn Consulting for the City of St. Helena. The interpreted location of the old dormant slide is approximately between 30 and 40 feet below the existing ground surface while the recent slide failure plane appears to pass just above the bottom of the spillway. The reviewer from USACE Los Angeles District questioned whether a failure plane exists where it is shown in cross-section A-A. Another interpretation of existing slides can be seen on Figure C-1 of the USACE Sacramento District report dated March 15, 2005. In this case, the predicted slide corresponding to a factor of safety of 1.0 is limited only in the road cut while the slides presented in cross- section A-A extend deeper and beyond the road and into the dam embankment. The difference in interpretation suggests that it is extremely difficult to predict past and future slides if one is to rely on limited boring information.

A layer of sediments exists upstream of the dam and varies along the channel alignment. The upstream toe of the dam is buried by the accumulated sediments. Generally the test pits upstream of the dam encountered predominantly fine-grained material (classified as sandy silt ML, and sandy silt with clay ML-CL under the Unified Soil Classification System) overlying coarse-grained materials consisting of sand and gravel (GP), silty gravel with sand (GP-GM), and sand with gravel (GP-SP).

This layer of accumulated sediments is estimated to be as much as 29 feet in thickness (Earth Imaging Geologic Services, 2005). At approximately 340 feet northwest of the upstream toe of dam, the sediment thickness grades down to 17 feet. The thickness of the sediments was based on the bedrock-sediment interface as surveyed using a Ground Penetrating Radar (GPR). However, the estimated sediment thickness does not make a distinction between the newly deposited sediments and the natural streambed prior to dam construction. While the 5 test pits provided information on material composition they were limited in depth due to caving of the soft to loose alluvial sediments. Thus, the recent accumulation of sediments could not be differentiated from the natural streambed material.

On the northeast side of Spring Mountain Road, exposures of welded tuff (Sonoma Volcanics) and serpentinite are clearly visible in the materials exposed by the recent landslide. Treadwell & Rollo (2003) reported that the Spring Mountain Road consisting of 6-inch thick asphalt pavement and 24-thick compacted road base appears to have been built upon a surface cut into the native serpentine bedrock material. This bedrock is typically dark greenish-gray, dark gray or black with occasional dark reddish-brown layers and is friable, deeply weathered, and intensely sheared. Corestones of harder, less weathered and less sheared serpentinite were present within the weaker matrix.

The streamwater is drained through a 6-feet diameter riser pipe (intake-outlet works) that is located approximately 40 feet northeast of the middle of the portion of the dam. The top of the pipe is at an elevation of 603.8 feet NGVD and is covered with a trash rack at the top. The outlet of the pipe is located at 160 feet downstream southeast at an elevation of 570.8 feet. The outlet pipe is connected to an 8 feet diameter stone culvert outfall structure at the downstream toe of the dam. According to preliminary estimate provided by USACE Water Resources Section, the most flow the riser pipe could handle would be about 50 cfs or less, which would generate a velocity of about 11 ft/sec at the bottom of the dam.

4.0 SLOPE STABILITY ANALYSIS

Subsurface conditions for slope stability and deformation analyses were derived from previous exploration projects. The dam site is underlain by fill overlying serpentinite and sheared shale that are prone to instability. Removal of the dam will likely result in creation of relatively steep side slopes rising up from the re-contoured channel thalweg to Spring Mountain Road on the east and to the adjacent hills on the west. The concern for instability of the excavated slope upon dam removal is the potential for adverse impact on Spring Mountain Road. In order to evaluate the risks associated with project alternatives a range of stability analyses was performed. For stability analysis, the required minimum factor of safety for short-term end-of-construction conditions is typically 1.3 and for long-term conditions, the minimum factor of safety is 1.5 (USACE EM 1110-2-1902, Slope Stability, October 31, 2003). For easy reference, Table 3-1 of EM 1110-2-1902 is repeated below.

Table 3-1 **Minimum Required Factors of Safety: New Earth and Rock- Fill Dams**		
Analysis Condition[1]	**Required Minimum Factor of Safety**	**Slope**
End-of-Construction (including staged construction) [2]	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool[3]	1.4	Downstream
Rapid drawdown	1.1-1.3[4,5]	Upstream
[1] For earthquake loading, see ER 1110-2-1806 for guidance. An Engineer Circular, "Dynamic Analysis of Embankment Dams," is still in preparation.		
[2] For embankments over 50 feet high on soft foundations and for embankments that will be subjected to pool loading during construction, a higher minimum end-of-construction factor of safety may be appropriate.		
[3] Pool thrust from maximum surcharge level. Pore pressures are usually taken as those developed under steady-state seepage at maximum storage pool. However, for pervious foundations with no positive cutoff steady-state seepage may develop under maximum surcharge pool.		
[4] Factor of safety (FS) to be used with improved method of analysis described in Appendix G.		
[5] FS = 1.1 applies to drawdown from maximum surcharge pool; FS = 1.3 applies to drawdown from maximum storage pool. For dams used in pump storage schemes or similar applications where rapid drawdown is a routine operating condition, higher factors of safety, e.g., 1.4-1.5, are appropriate. If consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered		

The initial analysis was performed by DWR in 2002. Due to the lack of an extensive subsurface investigation data, a rudimentary slope stability analysis was performed using Bishop's method of slices to compute the minimum factors of safety. Cohesion and friction angle values were estimated through back analysis of the "basis slope", i.e. assuming a factor of safety of 1 at the representative existing slope of 0.9 and 1.0. The back-calculated values were tested for 2 cross-sections identified as critical cross-sections. The range of back-calculated strength parameters varied from 0 psf cohesion with 34.1 degrees friction angle to 950 psf cohesion with 0 degrees friction angle. Through further evaluation, it was decided that the factors of safety were to be calculated using 300 psf and 26 degrees. Because of lack of site-specific data such as strength parameters in the above stability analyses the results should be viewed as qualitative. Nevertheless, the results of the analyses were evaluated by the USACE Sacramento District (SPK) to mean that: (1) the dam removal has less overall influence on the stability for larger and deeper slide planes, (2) smaller slides are more likely to occur than the relatively large scale slides. These are important findings or conclusion because they help to explain that even though smaller slides may occur, the overall stability of the site is less influenced by the proposed dam removal.

Recognizing the limitation of the DWR stability analyses the USACE Sacramento District performed a more expanded stability modeling utilizing the 2-dimensional computer program UTEXAS4 and the information provided in the Geotechnical Data Report by Treadwell & Rollo

(2003). The cross-section drawn along the crest of the dam at Station 20+20 with the presence of groundwater was used in the stability analyses.

The results of stability analysis are shown in **Appendix G4.** The calculated factors of safety range from less than 1.0 to as high as 2.2 depending on the material strength (residual vs. conservative strengths) and groundwater location (low or high). In general, the use of conservative strength values of 1000 psf and 37^0 resulted to factors of safety greater than 1.3 even in the presence of high water table. Conversely, a much lower factor of safety is computed for residual strengths of 600 psf and 18^0.

Based on the above, the USACE Sacramento District presented the following conclusions and recommendation:

(1) Removal of all or portion of the existing dam is expected to have no adverse impact on the road or left abutment concrete spillway. However, the spillway should be left in-place and backfilled to provide continued lateral support for the road.

(2) Minimize the excavation just to allow for adequate fish passage. Side slopes should be excavated no steeper than 1.5H:1V.

(3) Erosion protection should be placed at the toe of the new excavated slopes.

(4) Additional explorations and stability analyses for both large scale and small abutment slides are recommended for the final design.

(5) Implement an instrumentation program to monitor for slope movement.

5.0 DEFORMATION ANALYSIS

Following the stability analyses, it is recognized that even when the factors of safety appear to be within acceptable limit the ground area near the dam and spillway has apparently experienced some movement as evident from the lateral displacement of the spillway left chute wall (USACE site visit report, 2005). This observed movement is shown in **Figure 3.** It should

be noted that the movement of one spillway wall is not being used as evidence of a slope stability problem impacting Spring Mountain Road. The spillway wall has apparently experienced some movement since it was constructed. A steel supporting framework is acting against additional displacement. Unless a record of on-going monitoring measurements to support the observation, displacement could also have resulted from heavy road construction equipment working too close to the wall and other unknown site activities. Personal experience by the reviewer from the USACE Los Angeles District with retaining wall constructed for the Arizona Canal Diversion Channel in Phoenix, Arizona had similar deflection caused by water pushing between the wall and the backfill. These movements occurred during storm events when significant flows were going over the walls. The examples given here suggest that there are certainly a number of possibilities that could cause the spillway wall to move. A more accurate field procedure to measure the amount of lateral movement is through the use of an inclinometer. An inclinometer is a device made of flexible casing that is installed vertically into a borehole, with an inclinometer probe lowered into the casing to measure the lateral deformation of the ground. The monitoring of installed slope inclinometers may help to answer the question of current on-going movement. Preliminary baseline data from 3 inclinometers as well as groundwater measurements are provided in **Appendix G2**.

On the basis of the above preliminary field observation and prior documentation, a series of numerical analyses was undertaken to assist in the evaluation of ground movement or deformation during dam removal. The computer program PLAXIS version 8.2 (Brinkgreve, 2002) was used to simulate the removal of the dam and the subsequent deformation of the exposed slope face. The PLAXIS analysis was based on the cross section provided on Figure C-1 of the USACE Sacramento District report. This selected cross section is located at Station 20+20. A typical sequence of calculation phases is shown as follows.

Phase	Phase No.	Calculation Type	Load Input
Initial Phase	0		
Gravity	1	Plastic analysis	Total multiplier
Existing Condition	2	Plastic analysis	Staged construction
Backfill spillway	3	Plastic analysis	Staged construction
Excavate the dam & activate the first row of anchors	4	Plastic analysis	Staged construction
Excavate the dam & activate the second row of anchors	5	Plastic analysis	Staged construction
End of construction	6	Plastic analysis	Staged construction

The primary objective of these analyses was to estimate the deformations during the end-of-construction period, approximately 3 months. The necessary groundwater levels and slope reinforcement were also included in the deformation analyses. The data used in the analyses included the "conservative strength" values (i.e. cohesion: 1000 psf, friction angle: 37 degrees) used in the stability modeling by USACE Sacramento District.

In PLAXIS the deformation behavior of the excavated slope face was simulated using the Mohr-Coulomb Model. This model involves input parameters including friction angle, cohesion, stiffness, Poisson's ratio, and angle of dilatancy. The last three parameters were estimated based on their typical values. **Figure 4** shows the results of lateral deformation estimated from PLAXIS using the "conservative strengths". Results shown are for lateral deformations that would develop during the short-term end-of-construction period, which is generally considered to be a critical period.

6.0 GEOTECHNICAL COMPARISON OF ALTERNATIVES

The modeling of lateral deformations as presented in the Table below is preliminary as more data is needed to make the estimates more accurate. As such, the results of the modeling should be viewed as qualitative although a computational method is used. For comparison purposes, the range of lateral deformations are grouped arbitrarily in terms of high, medium and low deformations (see Note 2). **Figure 4** illustrates the modeling results for various dam configurations corresponding to the four sub-alternatives.

Alternative	**Side Slope**	**Estimated Lateral Deformation** [2]		**Remarks** [3]
	Left Abutment[1]	Without Reinforcement	With Reinforcement	
1A	1.5H:1V	High	Medium	Based on EOC
1B	0.8H:1V	High	Low	Based on EOC
2A	1.2H: 1V	Low	Low	Based on EOC
2B	1.5H:1V	Low	Medium	Based on EOC

Note: [1] Left abutment, adjacent to Spring Mountain Road and looking downstream. Right abutment side slope is maintained at 1.5H:1V.

[2] High: >12 inches; Medium: 6 to 11 inches; low: <5 inches

[3] EOC: End-of-construction condition.

Alternative 1A: Complete removal of dam and spillway

Alternative 1A is designed to be the most complete removal of the dam that includes the right wall of the spillway and all sediments behind the dam. The results show that very large lateral deformation should be expected when the dam is completely removed and bottom width of the channel is widened to as much as 53 feet. The amount of deformation is slightly reduced when rows of anchor piles are installed. This implies that up to a 29 foot wide bench and 23 feet wide channel could be constructed at the base of the excavation. In order to stabilize the toe of excavation, riprap protection would be necessary.

Alternative 1B: Removal of the dam while leaving spillway in place.

Alternative 1B involves removing the dam, the sediments behind the dam and keeping the spillway intact. In addition the spillway is backfilled to provide lateral support to the road. Given these the channel bottom can be constructed to a width of 53 feet at the expense of a steeper side slope (0.8H:1V) on the left abutment. The resulting deformation is quite high. To alleviate the large deformation a retaining wall is constructed adjacent to the spillway and a buttress support is provided at the base of the retaining wall at a slope of 1.5H:1V.

Alternative 2A: Notch Dam: Maximum notch size based on slope stability constraints and ecosystem goals.

Alternative 2A has a 32 foot wide bottom that includes a 9 foot-wide bench. It appears to be stable without slope stability measures. 2A also appears to be the most stable of all alternatives. Geotech recommends conservative planning due to several unknown factors and the lack of reliable and complete data. This recommendation is to include stability measures such as tiebacks and/or screw anchors to reduce slope failure risk. Geotech also recommends further investigation and the incorporation of new data from the piezometers and inclinometers that were installed in the fall of 2005 in order to better determine the need for these measures. The fact that the addition of measures decreases deformation can probably be explained though slight adjustments in modeling

- Width of total Channel: 32 feet
- Width of creek: 23 feet
- Width of bench: 9 feet

Alternative 2B: Notch Dam: Minimize notch size to the minimum hydrologic passage of 23 feet due to slope stability constraints.

Alternative 2B has a 23 foot wide bottom and no allowable space for a bench. The modeling results support SPK's recommendation favoring a notch alternative. This alternative also appears stable without geotechnical slope stability measures. However, Geotech recommends incorporating new data from piezometers and inclinometers to better determine the need for these recommended measures. Alternative 2B is the preferred geotechnical solution for reducing or removing barriers to fish passage and at the same time for maintaining a stable road. Under this alternative the spillway will remain in place and backfilled to provide continued support for the existing road.

- Width of total Channel: 23 feet
- Width of creek: 23 feet
- Width of bench: 0 feet

7.0 RECOMMENDATION

The following recommendations are made for the next phase of the project:

1. The soil parameters used in the analyses involve significant interpretation. Geotechnical data such as strength properties need to be refined and validated. It appears that a more detailed subsurface investigation and engineering are needed for the selected alternative during the Pre-construction Engineering and Design Phase (PED). The objective of the subsurface exploration is to obtain additional information that would allow us to evaluate the deformation characteristics and quality of the bedrock in-situ in relation to the stability and deformations of the excavated slope resulting from the removal of the dam embankment. The key to this assessment is measurement of deformations using in-situ testing methods such as pressuremeters; and performance of

continuous rock coring to aid in determining rock quality (RQD) and percent recovery at various depths and the presence of potential shear zones.

2. Maintaining the stability of the adjoining Spring Mountain Road is considered as a project constraint that must be addressed adequately to achieve project success. On this basis, a monitoring program should be implemented to quantify actual ground movement and stability at the site. The primary objective of the monitoring program is to obtain information that would allow us to evaluate the magnitude of deformations that may develop during and after removal of portion of the dam. The monitoring period will be for 6-month duration, which is typical for end-of-construction condition. The instrumentation program will include the following:

    - Inclinometers to measure lateral deformations near the excavated slope of the dam.

    - Piezometers or observation wells to measure the groundwater surface elevations.

3. Alternative 2B is the preferred geotechnical solution for maintaining stability and allowing for fish passage. It involves the following:

    - Removing a portion of the dam (notching) with 23 feet channel width.

    - Removing the sediments behind the dam.

    - Retain the spillway for maintaining support to the existing road.

Removing the inlet-outlet works consisting of a 6-feet diameter steel inlet pipe and possibly the stone culvert.

8.0 REFERENCES

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Brinkgreve, R.B.J. (2002). Editor: Plaxis: 2D-Version 8 – Finite Element Code for Soil and Rock Analyses. A.A. Balkema Publishers.

U.S. Army Corps of Engineers. 2003. Engineering Manual 1110-2-1902, Slope Stability, October 31, 2003.

U.S. Army Corps of Engineers, Sacramento District. 2005. Quality Control Plan for Geotechnical Products: Upper York Creek Dam Removal. March 15, 2005.

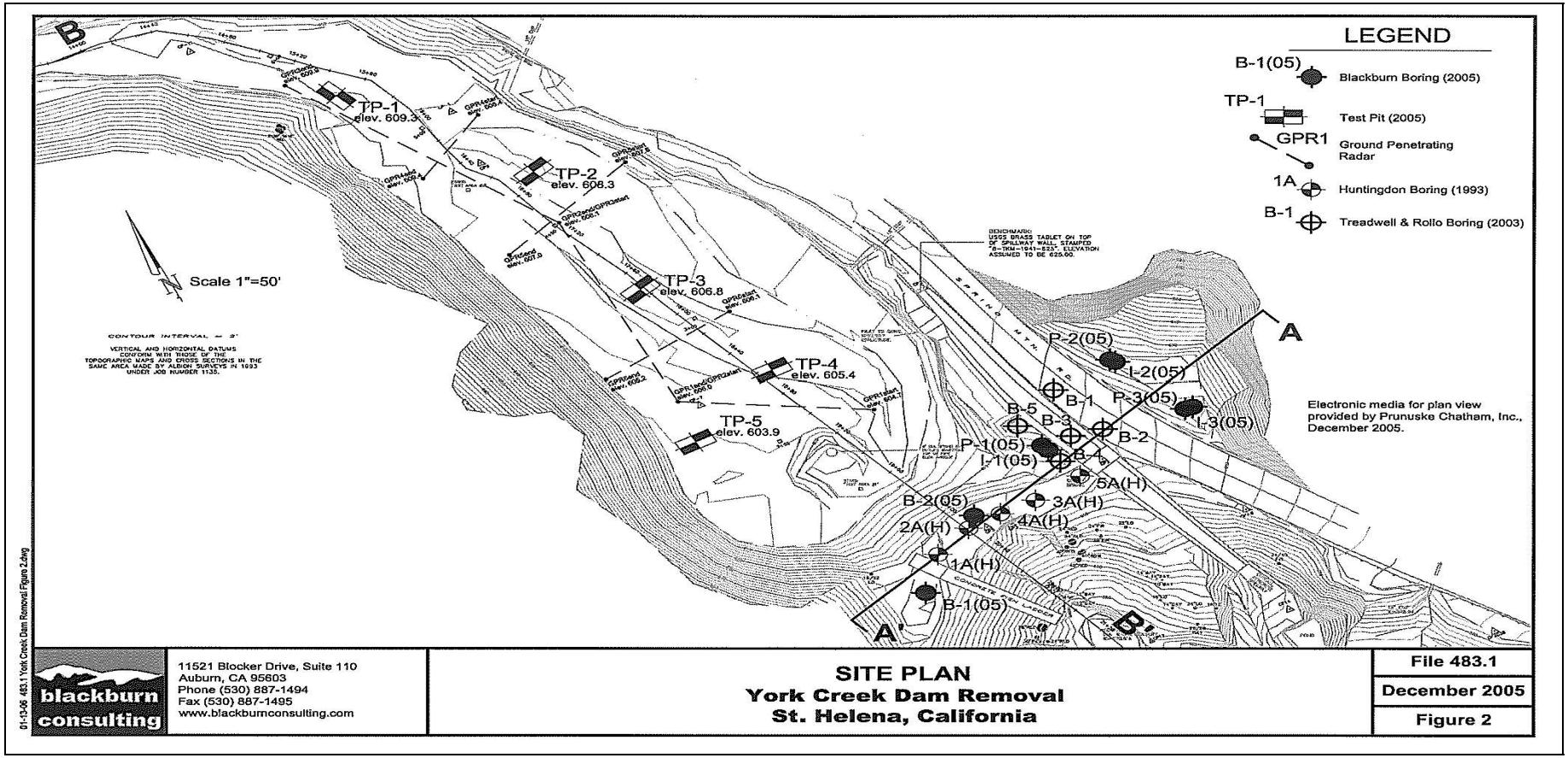


Figure 1. Exploration locations consisting of boreholes, test pits and ground penetrating radar (Reference: Blackburn Consulting Inc., 2006. Geotechnical Data Report prepared for the City of St. Helena).

US Army Corps of Engineers ®
San Francisco District

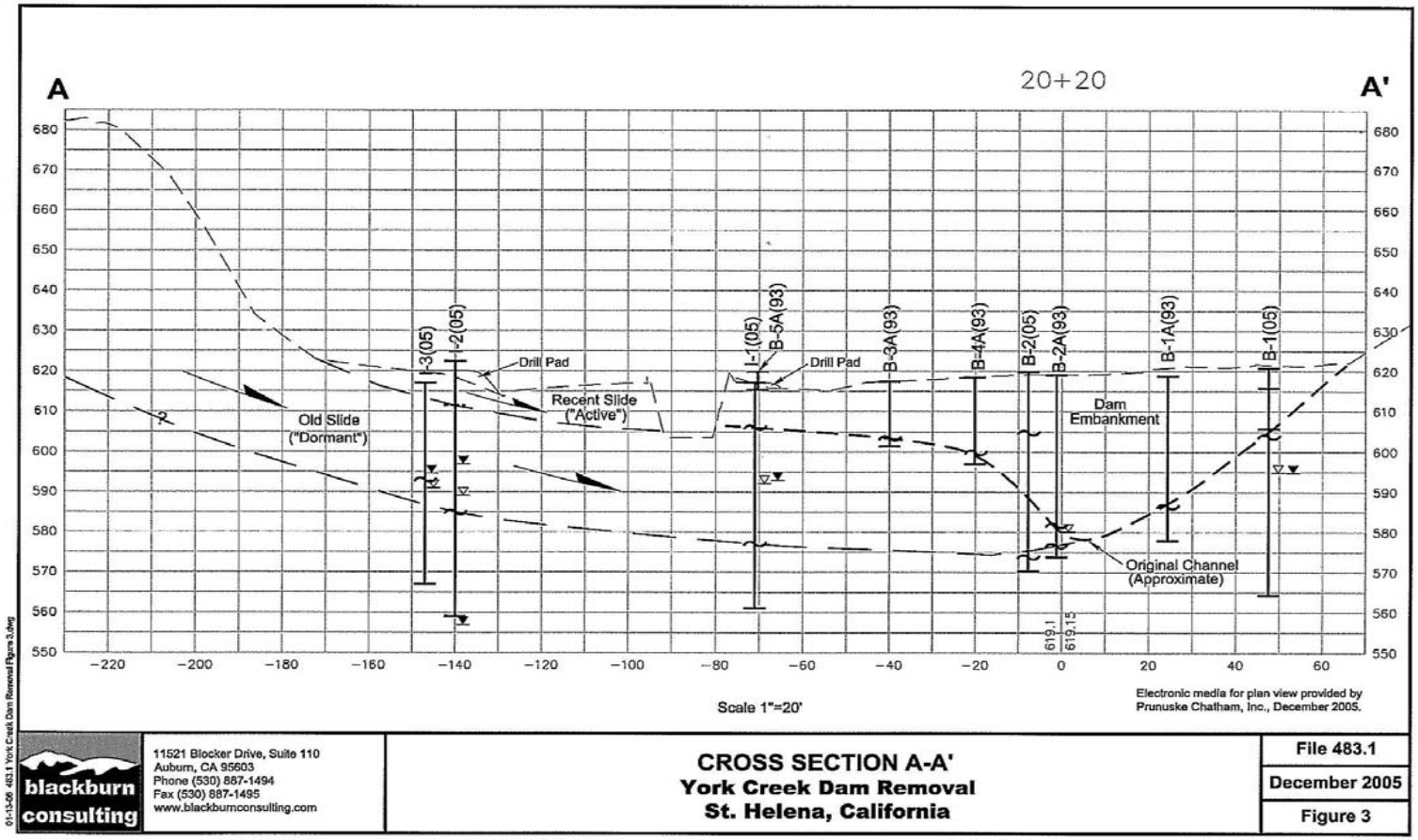


Figure 2. Transverse section along the dam embankment at Station 20+20. (Reference: Blackburn Consulting Inc. 2006. Geotechnical Data Report prepared for the City of St. Helena).

US Army Corps
of Engineers ®
San Francisco District

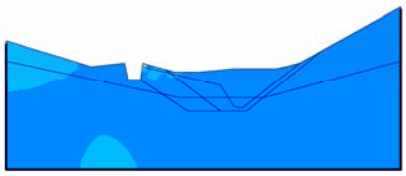
Figure 3. Observed spillway wall movement adjacent to Spring Mountain Road (November 17, 2005).

US Army Corps of Engineers ®
San Francisco District

UPPER YORK CREEK DAM REMOVAL AND ECOSYSTEM RESTORATION
St. Helena, California

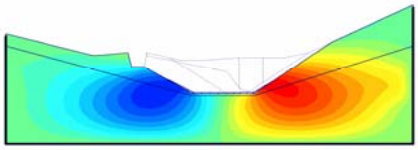
Geotechnical stability and deformation analyses

EXISTING CONDITION

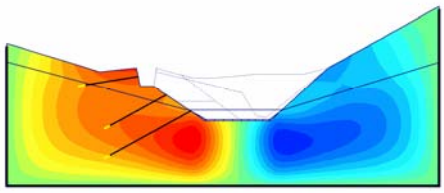


Factor of Safety: 2.9
Baseline lateral deformation: negligible

ALTERNATIVE 1A (Complete dam and spillway removal)

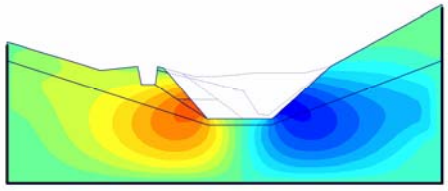


Lateral deformation: 29 inches
Measure: None

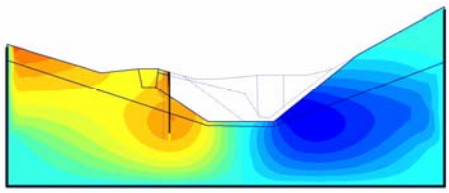


Lateral deformation: 11 inches
Measure: inclined screw anchors

ALTERNATIVE 1B (Complete dam removal, spillway intact)



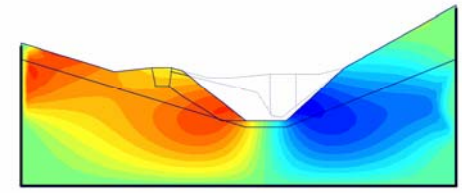
Lateral deformation: 36 inches
Measure: None



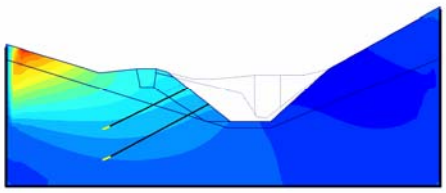
Lateral deformation: 3 inches
Measure: retaining wall

Figure 4. Results of Lateral Deformation Analysis. (Alternatives 1A and 1B)

ALTERNATIVE 2A (Notch Bottom Width > 23 ft)

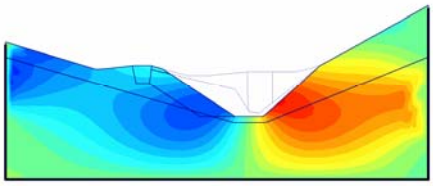


Lateral deformation: 5 inches
Measure: None

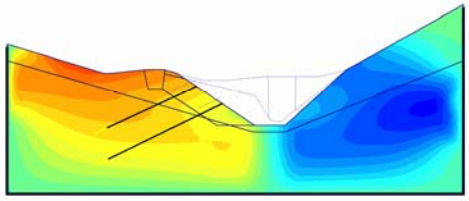


Lateral deformation: 4 inches
Measure: Inclined screw anchors

ALTERNATIVE 2B (Notch Bottom Width=23 ft)



Lateral deformation: 3 inches
Measure: None



Lateral deformation: 6 inches
Measure: inclined screw anchors

Figure 4. Results of Lateral Deformation Analysis. (Alternatives 2A and 2B)

APPENDIX G1

Boring Logs

LOG OF BORING B-1(O5)

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 10/25/05
DRILLING METHOD: 4" Solid Stem/Nx Rotary
LOGGED BY: BDC
CHECKED BY: RDS

ELEVATION: 620.8
DATUM:
WATER DEPTH: 19.5
READINGS TAKEN: 9:30am 10/25/05

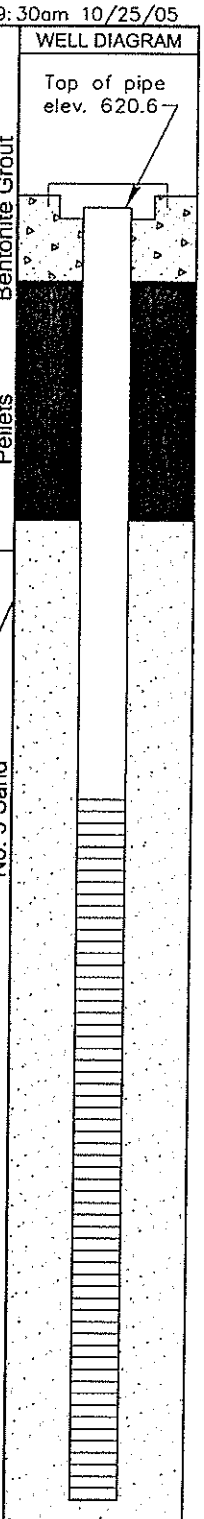
blackburn consulting

DIRECT SHEAR C (PCF)	DIRECT SHEAR φ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION
						5		B1-1	31		Gravelly Sand (SC-FILL): light gray, medium dense, dry, dam fill, gravel to 3" diameter (fractured green serpentinite)
					10.9			B1-2	16		
					14.4	10		B1-3	23		Clayey Sand with Gravel (SC-FILL): light gray, medium dense, dry, dam fill, gravel to 3" diameter (fractured green serpentinite)
					19.0	15		B1-4	14		Clayey Sand with Gravel (SC): green to light green, medium dense, dry to slightly moist, soft rock, serpentinite, original ground, completely decomposed ultramafic rock
					4.3	20		B1-5	33		Ultramafic Rock: green to light green, soft, moist, serpentinite mineral structure, very intensely weathered, friable, breaks with easy hand pressure
					19.0	25		B1-6	17		▽ 10/25/05
					14.1	30		B1-7	41		green, intensely to moderately weathered, dry
					21.3	35		B1-8	36		Switch to mud rotary drilling at 35.5ft. depth
					24.4	40		B1-9	62		soft rock, very intensely weathered, rock matrix is decomposing to clay
					16.5	45		B1-10	55		very dark green, moderately hard, moderately weathered, breaks with moderate hand pressure
					31.9	50		B1-11	50/0.4		moderately weathered, moderately hard
				119.2	15.8	55		B1-12	73		
						60					End of boring at 56.5ft. depth. Subsurface water encountered at 26.0ft. depth on 10/25/05.

GWS Measurements

Date	Time	Depth(ft)	GWS(elev.)
11/7/05	1200	26.4	594.2
11/9/05	1200	26.1	594.5

WELL DIAGRAM



01/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING B-2(05)

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 10/26&10/27/05
DRILLING METHOD: 4" Solid Stem/Nx Rotary
LOGGED BY: BDC/MDR
CHECKED BY: RDS

ELEVATION: 619.8
DATUM:
WATER DEPTH:
READINGS TAKEN:

DIRECT SHEAR C (PCF)	DIRECT SHEAR ϕ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION	WELL DIAGRAM
				114.7	10.3	5		B2-1	20		Sandy Clay with Gravel (CL-FILL): brown to dark brown, stiff to very stiff, dry, dam fill, gravel to 4" diameter	No Well Installed
				83.5	27.2	10		B2-2	20		very stiff, mixed with ultramafic gravel to 2" diameter (serpentinite rock)	
				74.4	36.2			B2-3	6		firm	
				74.9	30.5	15		B2-4	16		Ultramafic Rock: dark red and green, soft, friable, intensely weathered serpentinite, dam fill? Probable old landslide deposit easy drilling	
								B2-5	10			
				85.0	20.1	20		B2-6	21			
			0.6	92.5	55.3			B2-7	25			
			0.4	94.8	17.1	25		B2-8	15		Switch to mud rotary drilling at 24.5ft. depth	
				108.3	18.7			B2-9	19		soft, dark gray and brown, moist easy drilling	
				105.1	23.3	30		B2-10	19			
			0.3	115.9	24.2	35		B2-11	32		intensely weathered, dark gray/black, fractured, sheared	
				94.3	24.7	40		B2-12	16		moderately weathered rock	
			0.2	75.8	42.8	45		B2-13	34		Ultramafic Rock: hard; intact rock with rootlets at 46ft. depth	
				116.4	11.2			B2-14	67		harder drilling, moderately weathered	
						50					End of boring at 49.5ft. depth. No subsurface water encountered. Backfilled with cement/bentonite grout.	
						55						
						60						

01/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING P-1(O5)

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 11/08/05
DRILLING METHOD: 4" Solid Stem
LOGGED BY: MDR
CHECKED BY: RDS

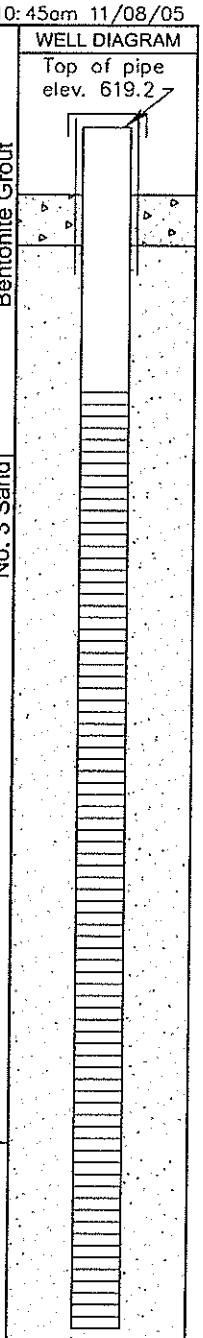
ELEVATION: 617.1
DATUM:
WATER DEPTH: 29.8
READINGS TAKEN: 10:45am 11/08/05

DIRECT SHEAR C (PCF)	DIRECT SHEAR φ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION
				90.7	13.4			P1-1	24		Sand with Silt and Gravel (SM/GM-FILL): tan/brown/gray/pink, medium dense, dry
				101.6	10.0	5		P1-2	12		
				84.4	16.1	10		P1-3	11		Ultramafic Rock: dark gray to brown, black and greenish-gray, soft to moderately hard, intensely weathered, damp to moist. (weathered Serpentine); probable old landslide deposit
			0.1	97.2	13.4	15		P1-4	28		
			0.3	81.9	18.6	20		P1-5	32		
			0.5	106.6	14.0	25		P1-6	18		▽ 11/08/05 moist to wet, with clay-filled seams
						30		P1-7	50/0.1		very hard augering
						35					easier augering
					12.8	40		P1-8	76/0.9		Ultramafic Rock: moderately to slightly weathered, hard; intact rock
						45					
						50					End of boring at 50.0ft. depth. Subsurface water encountered at 25.0ft. depth on 11/08/05.
						55					
						60					

GWS Measurements

Date	Time	Depth(ft)	GWS(elev.)
11/7/05	1200	24.2	595.0
11/9/05	1200	24.2	595.0

WELL DIAGRAM



01/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING P-2(O5)

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 10/27/05
DRILLING METHOD: 4" Solid Stem
LOGGED BY: MDR
CHECKED BY: RDS

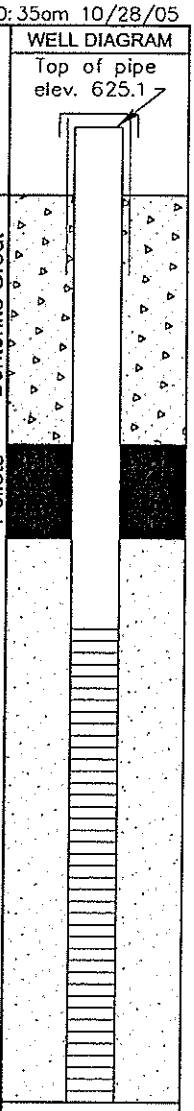
ELEVATION: 622.4
DATUM:
WATER DEPTH: 22.95
READINGS TAKEN: 10:35am 10/28/05

DIRECT SHEAR C (PCF)	DIRECT SHEAR φ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION
						0					Silty Sand with Gravel (SM/GM): tan to light brown, loose to medium dense, dry; recent landslide
						5					easier augering
						10					Sandy Clay (SC) with rock fragments: gray to brown, loose to medium dense, moist; Old landslide (?)
						15					easier augering
			0.8	106.4	15.6	20		P2-1	49		
						25					
						30					▽ 10/28/05
					RECOVERY (%)	35		P2-2	50/0.3		
					100			Run A			Ultramafic Rock (Serpentinite): dark green and gray-black, soft, intensely weathered/sheared, moist
					NR	40		Run B			slow, consistent coring
					20	45		Run C			easier coring from 45ft. to 48ft. depth
					100	50		Run D			harder coring below 48ft. depth
						55					End of boring at 52.3ft. depth. Subsurface water encountered at 33.3ft. depth on 10/27/05.
					10	60		Run C			

GWS Measurements

Date	Time	Depth(ft)	GWS(elev.)
11/7/05	1200	26.3	598.8
11/9/05	1200	25.5	599.6

WELL DIAGRAM



01/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING P-3(05)

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 11/07/05
DRILLING METHOD: 4" Solid Stem
LOGGED BY: MDR
CHECKED BY: RDS

ELEVATION: 616.7
DATUM:
WATER DEPTH: 28.5
READINGS TAKEN: 1:15pm 11/07/05

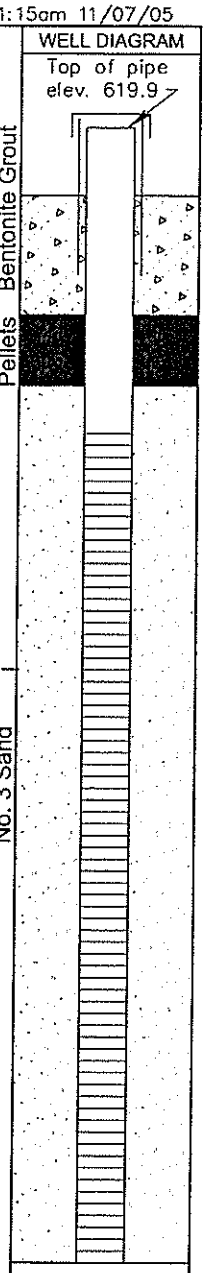
blackburn consulting

DIRECT SHEAR C (PCF)	DIRECT SHEAR ϕ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION
						0					Silty Sand with Gravel (SM/GM): tan to light brown, loose to medium dense, dry Recent landslide easy augering
						5					
						10					
						15					moderately hard augering --old landslide (?)
						20					Sandy Clay(SC) with rock fragments: gray to brown, loose to medium dense, moist ▽ 11/07/05 Old landslide (?)
						25					Ultramafic Rock (Serpentinite): dark green and gray-black, soft, intensely weathered/sheared, moist. Weathered in-place rock
						30					
						35					
						40					hard augering, serpentinite
						45					
						50					End of boring at 47.3ft. depth. Subsurface water encountered at 26.0ft. depth on 11/07/05.
						55					
						60					

GWS Measurements

Date	Time	Depth(ft)	GWS(elev.)
11/7/05	1200	22.3	597.6
11/9/05	1200	22.4	597.5

WELL DIAGRAM



01/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING I-1(05)

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 11/08/05
DRILLING METHOD: 4" Solid Stem
LOGGED BY: MDR
CHECKED BY: RDS

ELEVATION: 617.2
DATUM:
WATER DEPTH:
READINGS TAKEN:

blackburn consulting

DIRECT SHEAR C (PCF)	DIRECT SHEAR φ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION	WELL DIAGRAM
						0					Sand with Silt and Gravel (SM/GM-FILL): tan/brown/gray/pink, medium dense, dry	Top of pipe elev. 617.0
						5						Cement-Bentonite Grout
						10						
						15					Ultramafic Rock: dark gray to brown, black and greenish-gray, soft to moderately hard, intensely weathered, damp to moist. (weathered Serpentine); probable old landslide deposit	
						20						
						25					▽ 11/08/05 moist to wet, with clay-filled seams	
						30					very hard augering	
						35					easier augering	
						40					Ultramafic Rock: moderately to slightly weathered, hard; intact rock	
						45						
						50						
						55					End of boring at 56.0ft. depth. Subsurface water encountered at 25.0ft. depth on 11/08/05.	
						60						

0/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING I-2(05)

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 10/28/05
DRILLING METHOD: 4" Solid Stem
LOGGED BY: MDR
CHECKED BY: RDS

ELEVATION: 622.4
DATUM:
WATER DEPTH:
READINGS TAKEN:

DIRECT SHEAR C (PCF)	DIRECT SHEAR ϕ ANGLE°	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	RECOVERY (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION
						0					Silty Sand with Gravel (SM/GM): tan to light brown, loose to medium dense, dry; recent landslide
						5					easier augering
						10					Sandy Clay (SC) with rock fragments: gray to brown, loose to medium dense, moist; Old landslide (?)
						15					easier augering
						20					
						25					
						30					▽ 10/28/05
						35					
						40					Ultramafic Rock (Serpentinite): dark green and gray-black, soft, intensely weathered/sheared, moist; slow, consistent coring
						45					
					33	50		I2-1 Run A	22		easier coring from 45ft. to 48ft. depth; harder coring below 48ft. depth
					48	55		Run B			
					10	60		Run C			--CONTINUED--

WELL DIAGRAM

Top of pipe elev. 622.2

Cement-Bentonite Grout

0/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING I-2(O5) continued

FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 10/28/05
DRILLING METHOD: 4" Solid Stem
LOGGED BY: MDR
CHECKED BY: RDS

ELEVATION: 622.4
DATUM:
WATER DEPTH:
READINGS TAKEN:

DIRECT SHEAR C (PCF)	DIRECT SHEAR φ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	RECOVERY (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION	WELL DIAGRAM
					10			Run C			Ultramafic Rock (Serpentinite): dark green and gray-black, soft, intensely weathered/sheared, moist	Cement-Bentonite Grout
					NR			Run D				
						65					End of boring at 63.4ft. depth. Subsurface water encountered at 33.3ft. depth on 10/28/05.	
						70						
						75						
						80						
						85						
						90						
						95						

10/13/06 483.1 Boring Logs.dwg 1=1

LOG OF BORING I-3(05)

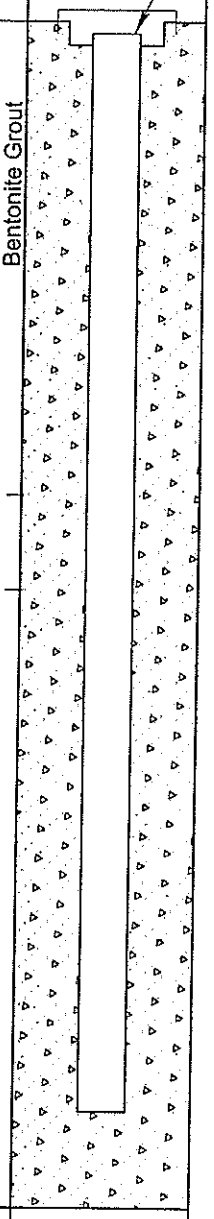
FILE No.: 483.1
PROJECT: York Creek Dam
LOCATION: St. Helena, California
CLIENT: City of St. Helena

DRILLING DATE: 11/07/05
DRILLING METHOD: 4" Solid Stem
LOGGED BY: MDR
CHECKED BY: RDS

ELEVATION: 617.0
DATUM:
WATER DEPTH:
READINGS TAKEN:

DIRECT SHEAR C (PCF)	DIRECT SHEAR φ ANGLE	% <200 SIEVE	UNCONFINED COMPRESSIVE STRENGTH (TSF)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DEPTH (FEET)	SAMPLE	SAMPLE No.	BLOW COUNT (N-Value)	GRAPHIC LOG	DESCRIPTION
						0					Silty Sand with Gravel (SM/GM): tan to light brown, loose to medium dense, dry Recent landslide easy augering
						5					
						10					
			0.6	136.8	20.0	15		I3-1	42		moderately hard augering --old landslide (?)
						20					Sandy Clay(SC) with rock fragments: gray to brown, loose to medium dense, moist ▽ 11/07/05 Old landslide (?)
			0.6	114.0	14.5	25		I3-2	82		Ultramafic Rock (Serpentinite): dark green and gray-black, soft, intensely weathered/sheared, moist. Weathered in-place rock
						30					
			1.9	102.7	21.9	35		I3-3	59		hard augering, serpentinite
						40					
				89.6	31.6	45		I3-4	60/0.5		
						50					End of boring at 50.0ft. depth. Subsurface water encountered at 23.0ft. depth on 11/07/05.
						55					
						60					

WELL DIAGRAM



0/13/06 483.1 Boring Logs.dwg 1=1

Appendix 2 Boring Logs from Treadwell & Rollo (2003)

Hole No. B-1

DRILLING LOG | DIVISION | INSTALLATION | SHEET 1 OF 3 SHEETS

1. PROJECT: UPPER YORK CREEK DAM, 3772.01	10. SIZE AND TYPE OF BIT: 3-7/8-inch carbide tooth 0.5 open bit
2. LOCATION (Coordinates or Station): Upstream end of spillway, east side (on Spring Mt. Rd.)	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): NGVD 29
3. DRILLING AGENCY: PC Exploration	12. MANUFACTURER'S DESIGNATION OF DRILL: CME 850 Track mounted and Mobile B-80 truck-mounted
4. HOLE NO. (As shown on drawing title and file number): B-1	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 3; UNDISTURBED 0
5. NAME OF DRILLER: John Stahlecker	14. TOTAL NUMBER CORE BOXES: 1
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN: 2.6	16. DATE HOLE: STARTED 10/6/2003; COMPLETED 10/9/2003
8. DEPTH DRILLED INTO ROCK: 30.4	17. ELEVATION TOP OF HOLE: +619.4 Feet
9. TOTAL DEPTH OF HOLE: 33.0	18. TOTAL CORE RECOVERY FOR BORING: 4.9/15.0 = 33 %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
+619.4 Feet	0.0		Asphalt		0.0	1300 hrs; begin drilling - mud rotary (water with Poly-Plus drill polymer)
	0.5				1.0	
			GRAVEL with SAND (GP) medium gray with orange brown mottling, dense, moist (ROAD BASE)	44	1	1305 hrs; resume drilling at 1325 hrs; CME autohammer, SPT sampler at 1-2.5 feet (blow counts every six inches = 10, 22, 24)
					1.0	
					2.5	
	2.5					smooth, even, slow drilling
			GRAVEL with SAND (GP) dark gray to dark greenish gray, very dense, moist, gravel is angular, fine to medium-grained serpentinite and possibly chert, with trace gray plastic clay [serpentinite]		2.5	1338 hrs; resume drilling at 1348 hrs
					3.0	
				22	2	SPT sampler at 3-4.5 feet (blow counts every six inches = 18, 26, 28)
					3.0	
					4.5	sample slipped out of SPT sampler, redrive sampler and recover 4 inches out of original 18-inch drive
	5.0				4.5	driller pulls bit; teeth are sheared off, possibly in asphalt, switch to tricone bit at 5 feet
					5.0	
			SERPENTINITE BEDROCK dark greenish gray with dark reddish brown layers, highly weathered, friable, soft, intensely sheared	61	3	1358 hrs; SPT sampler at 5-6.5 feet (blow counts every six inches = 8, 17, 23)
					5.0	resume drilling at 5 feet with tricone 1420 hrs
					6.5	
					6.5	lose some circulation at 7.5 feet
					8.5	
				100	4	1432 hrs; SPT sampler at 8-9.5 feet (blow counts every six inches = 20, 52, 58)
					8.0	
					9.5	resume drilling at 1448 hrs
					9.5	

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: +619.4 Feet

Hole No. B-1

PROJECT: UPPER YORK CREEK DAM, 3772.01

INSTALLATION:

SHEET 2 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SERPENTINITE BEDROCK dark greenish gray with dark reddish brown layers, highly weathered, friable, soft, intensely sheared *(continued)*		13.0	
				78	5 12.0 13.5	1504 hrs; hole caves at 8 feet when drillers insert sampler; SPT sampler 13-14.5 feet (blow counts every six inches = 10, 18, 18) redrill from 8 to 13 feet, 1601-1615 hrs
			becomes darker colored		14.5 18.0	1645 hrs; hole keeps caving in higher up when driller tries to obtain samples; driller said caving is likely to the coarse cuttings
				50	18.0 20.0	0908 hrs; 10/9/03-resume drilling with Mobile B-80 truck mounted drill loosing some drill water circulation into hole RQD = 0
				10	1 20.0 25.0	0943 hrs core from 20.0-25.0 feet; recovered as five small cobbles and some gravel RQD = 0

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: +619.4 Feet

Hole No. B-1

PROJECT: UPPER YORK CREEK DAM, 3772.01

INSTALLATION:

SHEET 3 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SERPENTINITE BEDROCK dark greenish gray with dark reddish brown layers, highly weathered, friable, soft, intensely sheared *(continued)*			
				50	1 25.0 28.0	0947 hrs; switch to diamond impregnate bit 1022 hrs core from 25.0-28.0 feet recovered as two small cobble-size pieces of core and lots of gravel, no recovery 26.5-28.0 feet due to gravels binding inside core barrel 1028 hrs
			becomes dark reddish brown			RQD = 0
				17	1 28.0 30.0	1039 hrs core from 28.0-30.0 feet recovered as one 2-inch piece of core, some gravel, and two small pieces of metal wedged in shoe, possibly from bit used from 0.0-5.0 feet 1045 hrs
						RQD = 0
				50	1 30.0 33.0	1114 hrs; resume after reaming hole
	33.0					
			Bottom of boring at 33 feet. Boring backfilled with grout on 10-9-03: 5% bentonite powder, 95% Portland I/II cement. No groundwater encountered during drilling.			1124 hrs

Hole No. B-2

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1 OF 3 SHEETS

1. PROJECT	UPPER YORK CREEK DAM, 3772.01
2. LOCATION (Coordinates or Station)	Down stream end of spillway, east side (on Spring Mt. Rd.)
3. DRILLING AGENCY	PC Exploration
4. HOLE NO. (As shown on drawing title and file number)	B-2
5. NAME OF DRILLER	John Stahlecker
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED ___ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	2.0
8. DEPTH DRILLED INTO ROCK	26.0
9. TOTAL DEPTH OF HOLE	28.0
10. SIZE AND TYPE OF BIT	HQ diamond impregnated
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	NGVD 29
12. MANUFACTURER'S DESIGNATION OF DRILL	Mobile B-80 Truck-mounted
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED 0 / UNDISTURBED 0
14. TOTAL NUMBER CORE BOXES	1
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 10/9/2003 / COMPLETED 10/9/2003
17. ELEVATION TOP OF HOLE	+616.9 Feet
18. TOTAL CORE RECOVERY FOR BORING	12.2/23.0 = 53 %
19. SIGNATURE OF INSPECTOR	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
+616.9 Feet	0.0		Asphalt			1256 hrs; begin drilling
	0.5				0.0	
			GRAVEL with SAND (GP) medium gray with brown mottling, dense, moist (ROAD BASE)		3.0	
	2.0		SERPENTINITE BEDROCK dark greenish gray to black, moderately to highly weathered, friable to weak, soft to moderately hard, highly fractured and sheared; rock exists as hard cobbles to boulder size rock in much softer, weaker, more weathered matrix.			driller set up new autohammer at 3 feet
				44	1	1302 hrs; SPT sampler, 3 to 4.5 feet (blow counts every six inches = 60, 46, 50)
					3.0	
					4.5	resume drilling 1342 hrs
					4.5	1345 hrs
			50 degree joint, stepped, rough, no infilling		5.0	
				47	1	1350 hrs
					5.0	
					8.0	
						RQD = 0
						1354 hrs; change to diamond surface set bit
				80	2	1407 hrs
					8.0	
					10.0	recovery is improved with surface set bit direct shear test and unconfined compression test performed, see Appendix E moisture content = 4.3 to 4.7 percent, dry density = 114 to 119 pcf
	10.0		becomes less weathered, weak, intensely fractured			RQD = 0

3891 psf
S_u = 1945 psf

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE +616.9 Feet | Hole No. B-2

PROJECT: UPPER YORK CREEK DAM, 3772.01 | INSTALLATION: | SHEET 2 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SERPENTINITE BEDROCK dark greenish gray to black, moderately weathered, weak soft to moderately hard, intensely fractured and sheared; rock, exists as gravel to boulder size rock in much softer weaker, more weathered matrix,	47	3 10.0 13.0	1414 hrs 1422 hrs RQD = 0 1430 hrs
				75	4 13.0 15.0	1438 hrs direct shear test and unconfined compression test performed, see Appendix E moisture content = 10.6 to 119 percent, dry density = 103 to 118 pcf RQD = 0 1443 hrs
	15.2		boulder (~15 to 15.5 feet bgs)	13	5 15.0 18.0	1510 hrs
			SERPENTINITE BEDROCK dark greenish gray to black, moderately to highly weathered, friable to weak, soft to moderately hard, intensely sheared			RQD = 0 1515 hrs
				90	6 18.0 20.0	1524 hrs RQD = 0 1532 hrs
				90	7 20.0 23.0	1539 hrs direct shear test and unconfined compression test performed; see Appendix E moisture content = 3.6 to 8.1 percent, dry density = 103 to 126 pcf RQD = 0

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: +616.9 Feet

Hole No. B-2

PROJECT: UPPER YORK CREEK DAM, 3772.01

INSTALLATION:

SHEET 3 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SERPENTINITE BEDROCK dark greenish gray to black, moderately to highly weathered, friable to weak, soft to moderately hard, intensely sheared *(continued)*	40	8 23.0 25.0	1547 hrs 1556 hrs RQD = 0
			becomes less weathered, weak, intensely fractured	20	9 25.0 28.0	1612 hrs 1602 hrs RQD = 0
	28.0					1619 hrs
			Bottom of boring at 28.0 feet. Boring backfilled with grout on 10-9-03: 5% bentonite powder, 95% Portland I/II cement. Groundwater measured at 14.5 feet bgs (Elevation 602.2 feet on 10-9-03 at 1745 hrs).			

Hole No. B-3

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1 OF 2 SHEETS

1. PROJECT: UPPER YORK CREEK DAM, 3772.01	10. SIZE AND TYPE OF BIT: HQ diamond impregnated, face discharge
2. LOCATION (Coordinates or Station): Center of spillway chute	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): NGVD 29
3. DRILLING AGENCY: PC Exploration	12. MANUFACTURER'S DESIGNATION OF DRILL: Deitrich D-25 skid-mounted
4. HOLE NO. (As shown on drawing title and file number): B-3	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 0; UNDISTURBED 0
5. NAME OF DRILLER: John Stahlecker	14. TOTAL NUMBER CORE BOXES: 1
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED ___ DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN: 6.0	16. DATE HOLE: STARTED 10/8/2003; COMPLETED 10/8/2003
8. DEPTH DRILLED INTO ROCK: 14.5	17. ELEVATION TOP OF HOLE: +603.7 Feet
9. TOTAL DEPTH OF HOLE: 15.0	18. TOTAL CORE RECOVERY FOR BORING: 8.2/14.9 + 55 %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
+603.7 Feet	0.0		Concrete			0828 hrs; begin coring concrete floor of spillway
	0.5					0832 - 0844 hrs; stop to fix bolt on pull-down cable
			SERPENTINITE BEDROCK dark green, black, and dark reddish brown, moderately to highly weathered, soft to moderately hard, friable to moderately strong, highly fractured and sheared; rock exists as gravel to boulder size rock in much softer, weaker, more weather matrix	22	1 0.5 5.0	0854 hrs; drill is through concrete 0915 hrs; begin coring below the spillway concrete; using water only, no polymer
						losing drill water circulation ~5 gpm
						losing about 10 gpm RQD = 0
						0932 hrs
				100	2 5.0 6.5	0941 hrs
			two joints at 25 and 45 degrees, 1/16-inch-wide, filled with greenish gray mineralization			RQD = 23% water loss stops ~6.0 feet, (likely fractures became plugged) 0951 hrs
				80	3 6.5 8.0	1000 hrs
						RQD = 33%
						1010 hrs
			45 degree, 1/8-inch-wide, filled with gray mineralization	100	4 8.0 9.5	1033 hrs
						hard slow drilling RQD = 87%
			40 degree, 1/8-inch-wide, filled with gray and white mineralization			1042 hrs
				57	5 9.5	1100 hrs

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: +603.7 Feet

Hole No. B-3

PROJECT: UPPER YORK CREEK DAM, 3772.01

INSTALLATION:

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SERPENTINITE BEDROCK dark green, black, and dark reddish brown, moderately to highly weathered, soft to moderately hard, friable to moderately strong, highly fractured and sheared; rock exists as gravel to boulder size rock in much softer, weaker, more weather matrix *(continued)* 45 degree, 1/8-inch-wide, filled with gray and green mineralization		12.5	RQD = 11% 1242 hrs
			45 degree, 1/8-inch-wide, filled with gray and green mineralization highly fractured zones highly fractured zones 60 degree joint, slikensides	100	6 12.5 15.0	1233 hrs RQD = 0
	15.0		Bottom of boring at 15.0 feet on 10-8-03. Hole caved in to 4.5 feet after coring equipment removed from boring. Backfilled with bentonite chips and Portland I/II cement. No groundwater encountered during drilling.			hard, slow drilling from 14.0 feet to bottom 1303 hrs

Hole No. B-4

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1 OF 4 SHEETS

1. PROJECT	UPPER YORK CREEK DAM, 3772.01
2. LOCATION (Coordinates or Station)	Down stream end of spillway, west side of spillway
3. DRILLING AGENCY	PC Exploration
4. HOLE NO. *(As shown on drawing title and file number)*	B-4
5. NAME OF DRILLER	John Stahlecker
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	20.2
8. DEPTH DRILLED INTO ROCK	15.8
9. TOTAL DEPTH OF HOLE	36.0
10. SIZE AND TYPE OF BIT	HQ diamond impregnated
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*	NGVD 29
12. MANUFACTURER'S DESIGNATION OF DRILL	CME 850 Track mounted
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED 9 / UNDISTURBED 0
14. TOTAL NUMBER CORE BOXES	1
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 10/10/2003 / COMPLETED 10/10/2003
17. ELEVATION TOP OF HOLE	+617.9 Feet
18. TOTAL CORE RECOVERY FOR BORING	11.8/16.0 = 74 %
19. SIGNATURE OF INSPECTOR	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+617.9 Feet	0.0		SANDY SILT (ML)/SILTY SAND (SP) brown, medium stiff/loose, dry [FILL]	67	1 0.0 1.5	S&H sampler from 0 to 1.5 feet (blow counts every six inches = 8, 11,13)
	1.5		GRAVELLY CLAY (CL) gray/green, stiff, moist [FILL]	83	2 1.5 3.0	SPT sampler from 1.5 to 3 feet (blow counts every six inches = 12, 4, 5)
	3.0		SANDY CLAY (CL) mottled dark brown, green-gray, and light gray, medium stiff, moist, with serpentinite and welded tuff cobbles [FILL]	100	3 3.0 5.0	coring begins at 3.0 feet (soil core not saved; only saved rock below 20.5 feet)
				10	4 5.0 7.5	SPT sampler at 5-6.5 feet (blow counts every six inches = 4, 4, 4)
				20	5 7.5 10.0	

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: +617.9 Feet

Hole No. B-4

PROJECT: UPPER YORK CREEK DAM, 3772.01

INSTALLATION:

SHEET 2 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SANDY CLAY (CL) mottled dark brown, green-gray, and light gray, medium stiff, moist, with serpentinite and welded tuff cobbles [FILL] *(continued)*	83	. 6 10.0 11.5	1407 hrs; SPT sampler at 10-11.5 feet (blow counts every six inches = 9, 8, 12)
			serpentinite cobbles (~11-11.5 bgs)	93	7 11.5 15.0	
	12.5		CLAYEY GRAVEL (GP) grayish dark green, medium dense, moist, angular gravels in sandy clay matrix (CRUSHED SERPENTINITE) [FILL]			RQD = 0
			one-inch brown sandy clay seam	100	8 15.0 16.5	SPT sampler at 15-16.5 feet (blow counts every six inches = 6, 7, 8)
				29	9 16.5 20.0	1432 hrs RQD = 0
	20.3		SERPENTINITE BEDROCK dark greenish gray to black, moderately to highly weathered, friable to weak, soft to moderately hard, highly fractured and sheared; rock exists as gravel to boulder size rock in much softer, weaker, more weathered matrix	87	10 20.0 21.5	1442 hrs; SPT sampler at 20-21.5 feet (blow counts every six inches = 5, 7, 8) 1450 hrs
				77	1 21.5 25.0	1457 hrs direct shear test performed, see Appendix E moisture content = 30.0 to 30.4 percent, dry density = 90 to 93 pcf RQD = 0

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE +617.9 Feet | Hole No. B-4

PROJECT: UPPER YORK CREEK DAM, 3772.01 | INSTALLATION | SHEET 3 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SERPENTINITE BEDROCK dark greenish gray to black, moderately to highly weathered, friable to weak, soft to moderately hard, highly fractured and sheared; rock exists as gravel to boulder size rock in much softer, weaker, more weathered matrix *(continued)* becomes mottled with rust brown staining			
				100	11 25.0 26.5	SPT sample at 25-26.5 feet (blow counts every six inches = 8, 10, 12) moisture content = 62.9 percent
				95	1 26.5 30.0	1524 hrs unconfined compression test and direct shear test performed, see Appendix E moisture content = 26.1 to 38.8 percent, dry density =75 to 92 pcf RQD = 0
				73	1 30.0 34.5	1544 hrs RQD = 0
			hard boulder	100	12 34.5 36.0	1600 hrs; SPT sampler at 34.5 -36 feet (blow counts every six inches = 8, 21, 22) direct shear test performed see Appendix E moisture content = 19.3 percent, dry density = 106 pcf

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: +617.9 Feet

Hole No. B-4

PROJECT: UPPER YORK CREEK DAM, 3772.01

INSTALLATION:

SHEET 4 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
	36.0					1622 hrs
			Bottom of boring at 36.0 feet on 10-10-03. Boring backfilled with grout on 10-13-03: 5% bentonite powder 95%, Portland I/II cement. Groundwater encountered at 24.6 feet depth (Elevation 593.3 feet) on 10-13-03.			

Hole No. B-5

DRILLING LOG | DIVISION | INSTALLATION | SHEET 1 OF 4 SHEETS

1. PROJECT: UPPER YORK CREEK DAM, 3772.01	10. SIZE AND TYPE OF BIT: HQ diamond impregnated
2. LOCATION (Coordinates or Station): Upstream end of spillway, west side of spillway	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): NGVD 29
3. DRILLING AGENCY: PC Exploration	12. MANUFACTURER'S DESIGNATION OF DRILL: CME 850 Track mounted drill
4. HOLE NO. (As shown on drawing title and file number): B-5	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 7; UNDISTURBED 0
5. NAME OF DRILLER: John Stahlecker	14. TOTAL NUMBER CORE BOXES: 1
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED ___ DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN: 13.2	16. DATE HOLE: STARTED 10/13/2003; COMPLETED 10/13/2003
8. DEPTH DRILLED INTO ROCK: 23.3	17. ELEVATION TOP OF HOLE: +620.5 Feet
9. TOTAL DEPTH OF HOLE: 36.5	18. TOTAL CORE RECOVERY FOR BORING: 7.5/17.5 43 %
	19. SIGNATURE OF INSPECTOR

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
+620.5 Feet	0.0		SILTY, CLAYEY SAND (SC-SM) brown, medium dense, moist, with serpentinite and tuff cobbles [FILL]	66	1 0.0 1.5	0920 hrs; begin drilling S&H sampler at 0-1.5 feet (blow counts every six inches = 42, 43, 27) tip of S&H sampler blocked with serpentinite cobble gave unrealistically high blow counts
				80	2 1.5 3.0	SPT sampler, at 1.5 to 3 feet (blow counts every six inches = 9, 7, 5) losing small amount of water into hole
	3.5		CLAYEY SAND (SC) dark brown and dark greenish gray, loose to medium dense, moist, with crushed highly weathered serpentinite [FILL]			begin core drilling with water and polymer - only saved rock core below 16.5 feet 0955 hrs
				66	3 5.0 6.5	S&H sampler, at 5-6.5 feet (blow counts every six inches = 3, 6, 7) 1005 hrs; resume coring
				80	4 6.5 8.0	SPT sampler at 6.5-8 feet (blow counts every six inches = 3, 5, 5)
				75	5 8.0 10.0	
	9.0		GRAVELLY CLAY (CL) grayish dark green, very stiff, moist, angular gravels and cobbles in sandy clay matrix (CRUSHED SERPENTINITE) [FILL]			1020 hrs

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE +620.5 Feet | Hole No. B-5

PROJECT: UPPER YORK CREEK DAM, 3772.01 | INSTALLATION: | SHEET 2 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			GRAVELLY CLAY (CL) grayish dark green, very stiff, moist, angular gravels and cobbles in sandy clay matrix (CRUSHED SERPENTINITE) [FILL] *(continued)*	100	5 10.0 11.5	S&H sampler, at 10-11.5 feet (blow counts every six inches = 16, 23, 19) 1040 hrs; resume drilling Note: sampler has several weathered serpentinite cobbles so blow counts may not represent true density of fill
				73	7 11.5 13.0	SPT sampler at 11.5-13 feet (blow counts every six inches = 5, 6, 8) still losing small amount of water into hole ~1-2 gpm
	13.3		SERPENTINITE BEDROCK grayish dark green, friable, moderate to highly weathered, soft; rock exists as harder corestone in much softer, weaker, more weathered matrix, highly fractured and sheared	75	8 13.0 15.0	RQD = 0 1054 hrs; SPT sampler at 15-16.5 feet (blow counts every six inches = 8, 11, 14) 1120 resume drilling
				100	9 15.0 16.5	
				91	1 16.5 20.0	RQD = 0 1140 hrs; SPT sampler at 20-21.5 feet (blow counts every six inches = 12, 14, 17) 1233 hrs
				93	10 20.0 21.5	moisture content = 24.2 percent
				23	1 21.5 25.0	RQD = 0 no recovery 22.3 to 25.0 in clayey softer

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE +620.5 Feet | Hole No. B-5

PROJECT: UPPER YORK CREEK DAM, 3772.01 | INSTALLATION: | SHEET 3 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
			SERPENTINITE BEDROCK grayish dark green, friable, moderate to highly weathered, soft; rock exists as harder corestone in much softer, weaker, more weathered matrix, highly fractured and sheared *(continued)*			zones
				58	1 25.0 27.5	1255 hrs
			vertical slickensided fracture runs length of recovered core from 25.0 to 26.4 feet			RQD = 0
						1304 hrs
				0	1 27.5 30.0	1313 hrs no recovery 27.5 to 30 feet, core completely washed away RQD = 0
						1318 hrs; SPT sampler at 30-31.5 feet (blow counts every six inches = 16, 14, 17)
				80	11 30.0 31.5	pull rods and check the waterways on bit - all are blocked with clay - driller cleans them out
				60	1 31.5 32.5	RQD = 0
						1351 hrs
				0	1 32.5 35.0	1403 hrs RQD = 0
						1425 hrs
				67	12 35.0 36.5	S&H sampler at 35-36.5 feet (blow counts every six inches = 10, 18, 37) unconfined compression test

ENG FORM JUN 67 1836-A

PROJECT: UPPER YORK CREEK DAM, 3772.01 | HOLE NO. B-5

S_u = 2264 psf

S_u = 1131 psf

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: +620.5 Feet

Hole No. B-5

PROJECT: UPPER YORK CREEK DAM, 3772.01

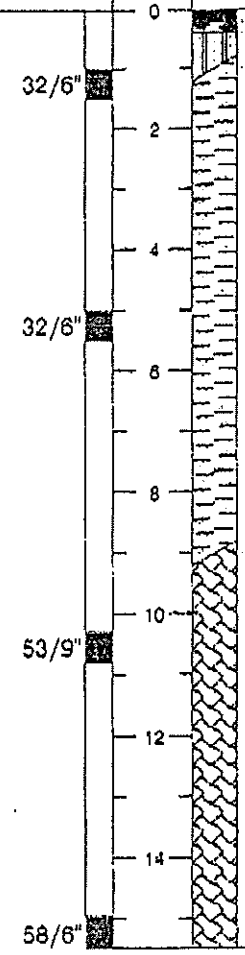
INSTALLATION:

SHEET 4 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
						performed, see Appendix E moisture test = 10.9 percent dry density = 105 pcf
	36.5					1425 hrs
			Bottom of boring at 36.5 feet, 10-13-03 Boring backfilled with grout on 10-13-03: 5% bentonite 95% Portland cement. No groundwater encountered during drilling.			

Appendix 3 Boring Logs from Huntingdon/ Herzog Associates Inc. (1993)

Other Laboratory Tests	Dry Density (pcf)	...sture Content (%)	Blows/ Foot *	DEPTH (FEET)	EQUIPMENT: 6" Fli' ...t Auger	ELEVATION: 527' **
					LOGGED BY: J. Noble	START DATE: 7-22-93
						FINISH DATE: 7-22-93



2" ASPHALT

2" BASEROCK

BROWN SILTY SAND (SM)
medium dense, moist (Fill)

WHITE TUFF, extremely closely spaced fractures, weak, friable, highly weathered

becoming brown at 4 feet

becoming white at 5 feet

grading back into brown tuff at 8 feet

GRAY-GREEN SERPENTINITE, extremely closely spaced fractures, weak, friable, highly weathered

becoming plastic, highly weathered at 14 feet

BOTTOM OF BORING 1 @ 15.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Reference: Approximate elevations based on Topographic Map by Albion Surveys dated June 1993.

Huntingdon
Herzog Associates, Inc.

Job No: 6636.01 00.1
Appr:
Drwn: PD
Date: AUG 1993

LOG OF BORING 1

UPPER RESERVOIR

St. Helena Dam

PLATE 2

EQUIPMENT: 6" [illegible]ht Auger
LOGGED BY: J. Noble

ELEVATION: 623' **
START DATE: 7-22-93
FINISH DATE: 7-22-93

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	2" ASPHALT 2" BASEROCK RED-BROWN GRAVELLY SILT (ML) stiff, moist (Fill)
			27	2	
				4	BROWN-GREEN SILTY GRAVEL (GM) stiff, moist (Native)
			28	6	
				8	WHITE TUFF, extremely closely spaced fractures, weak, friable, highly weathered
			24	10	
				12	
				14	becoming less fractured at 14 feet
			54/ 11"	16	
			49	18	BOTTOM OF BORING 2 @ 18.5 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Reference: Approximate elevations based on Topographic Map by Albion Surveys dated June 1993.

Huntingdon
Herzog Associates, Inc.

Job No: 6636.01.00.1
Appr:
Drwn: PD
Date: AUG 1993

LOG OF BORING 2

UPPER RESERVOIR

St. Helena Dam

PLAT 3

EQUIPMENT: 6" F ht Auger
LOGGED BY: J. Noble

ELEVATION: 686' **
START DATE: 7-27-93
FINISH DATE: 7-27-93

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	BROWN SAND(SP) medium dense, dry, becoming wet at 1.5'
				2	BROWN TO GRAY SANDY SILT(MH) soft, saturated
			10	4	
				6	occasional lenses of sands and gravels, heaving sands, occasional large gravels
				8	
				10	GRAY INTERLAYERED SANDY SILT(MH), SANDY GRAVEL(SP), medium stiff to loose, saturated
SA, see Plate 11			9	12	gravels to 17 feet GRAY SILTY SAND(SM) loose, saturated, with organics
			32/2"	14	GRAY SANDY GRAVEL, dense, saturated, with cobbles (old stream bed)
				16	BLUE-GREEN SERPENTINITE, extremely closely spaced fractures, weak, friable, highly weathered
			50/6"		BOTTOM OF BORING 3 @ 17.5 FEET

* Converted to equivalent standard penetration blow counts.

** Reference: Approximate elevations based on Topographic Map by Albion Surveys dated June 1993.

Huntingdon
Herzog Associates, Inc.

Job No: 6636.01.00.1
Appr:
Drwn: PD
Date: AUG 1993

LOG OF BORING 3

UPPER RESERVOIR

St. Helena Dam

PLATE 4

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	BROWN SANDY GRAVEL(GP) medium dense, moist
				2	BROWN SILT (ML) soft, wet, (Fill) smooth drilling at 3'
SA, see Plate 11			3	4	
				6	gravels at 6'-6.5' (Fill)
				8	BROWN SANDY CLAY(CL) stiff, wet
					Groundwater at 10 feet
			11	10	GRAY-GREEN SANDY GRAVEL (GC) medium dense, saturated (Possibly Slide Debris)
				12	very hard drilling
			50/6"		heaving gravels and cobbles
				14	drilling smoothes out at 14.5'
			50/2"		WHITE-GRAY PYROCLASTIC TUFF,closely spaced fractures, weak to moderately strong, highly weathered
					BOTTOM OF BORING 4 @ 15.5 FEET

EQUIPMENT: 6" r ht Auger
LOGGED BY: J. Noble
ELEVATION: ' **
START DATE: 7-27-93
FINISH DATE: 7-27-93

* Converted to equivalent standard penetration blow counts

** Reference: Approximate elevations based on Topographic Map by Albion Surveys dated June 1993.

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Herzog Associates, Inc.

Job No: 6636.01.00.1
Appr:
Drwn: PD
Date: AUG 1993

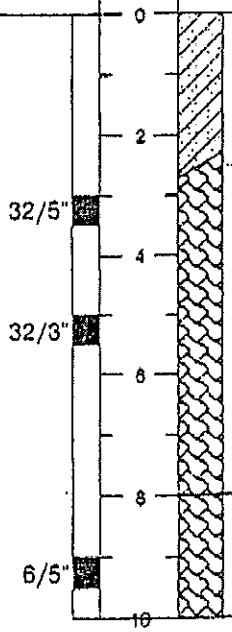
LOG OF BORING 4

UPPER RESERVOIR

St. Helena Dam

PLATE 5

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	EQUIPMENT: 8" Fl[illegible] Auger	ELEVATION: ' **
					LOGGED BY: J. Noble	START DATE: 7-27-93
						FINISH DATE: 7-27-93



BROWN GRAVELLY CLAYEY SAND (SC)
medium stiff, moist (Fill)

BROWN SERPENTINITE, extremely closely spaced fractures, weak, friable, highly weathered

easy drilling at 7'-8'

becoming gray-green in color at 8 feet

BOTTOM OF BORING 5 @ 10 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Reference: Approximate elevations based on Topographic Map by Albion Surveys dated June 1993.

Huntingdon
Herzog Associates, Inc.

Job No: 6636.01.00.1
Appr:
Drwn: PD
Date: AUG 1993

LOG OF BORING 5

UPPER RESERVOIR

St. Helena Dam

PLATE 6

EQUIPMENT: 6" Fl[illegible] Auger
LOGGED BY: J. Noble
ELEVATION: [illegible] **
START DATE: 7-27-93
FINISH DATE: 7-27-93

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	BROWN GRAVELLY CLAY (CL) stiff, moist, with occasional large cobbles (Fill)
				2	GRAY CLAYEY SAND (SC) medium dense, moist (Fill)
				4	becoming wet at 4 feet
			11		
				6	BROWN SANDY CLAY (CL) medium stiff, wet (Fill)
				8	
			18		
				10	
TxUU, see Plate 10	82	30.6	15		
				12	
					DARK GRAY SANDY CLAY(CL) medium stiff, wet, with gravels
				14	
					GREEN SERPENTINITE, extremely closely spaced fractures to sheared, weak, friable, highly weathered
			17	16	
				18	
			38/6"	20	
					BOTTOM OF BORING 6 @ 21 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Reference: Approximate elevations based on Topographic Map by Albion Surveys dated June 1993.

Huntingdon
Herzog Associates, Inc.

Job No: 6636.01.00.1
Appr:
Drwn: PD
Date: AUG 1993

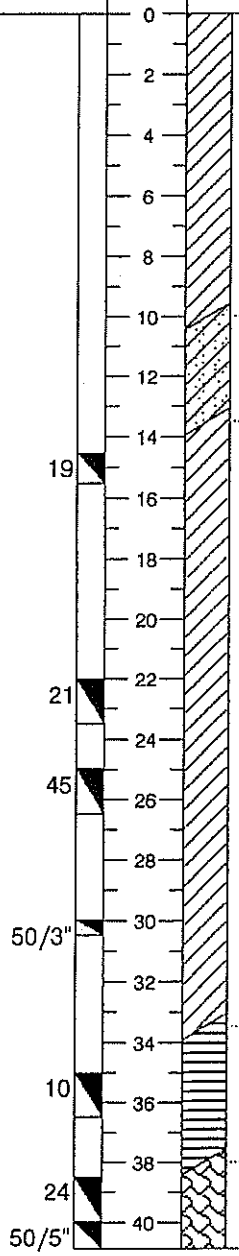
LOG OF BORING 6
UPPER RESERVOIR
St. Helena Dam

PLATE 7

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)

EQUIPMENT: 8" Ho / Auger
LOGGED BY: J. Noble

ELEVATION: 619.0' **
START DATE: 11-17-93
FINISH DATE: 11-17-93



BROWN SANDY CLAY(CL)
medium stiff to soft, moist to wet, with variable gravel and cobble content (Fill)

becoming stiffer below 8'

LIGHT BROWN TO GRAY CLAYEY SAND(SC)
medium dense, moist (Fill)

GREEN TO DARK GRAY SANDY CLAY(CL)
medium stiff to soft, with occasional gravels and wood (Fill)

harder drilling at 25 feet
occasional serpentinite gravels

large redwood root

BLUE-GRAY SHALE AND SERPENTINITE, shattered, weak, friable, highly weathered

GREEN SERPENTINITE, shattered, weak, friable, highly weathered

BOTTOM OF BORING 1A @ 40.9 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Approximate ground surface at time of drilling.

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Job No: 6636.01.01.1
Appr: CH
Drwn: PD
Date: NOV 1993

LOG OF BORING 1A

UPPER RESERVOIR
St. Helena, California

PLATE 2

EQUIPMENT: 8" Hol ' Auger
LOGGED BY: J. Noble

ELEVATION: 617.0' **
START DATE: 11-17-93
FINISH DATE: 11-17-93

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	BROWN GRAVELLY CLAY(CL) medium stiff, moist to wet, with abundant gravels (Fill)
				2	
				4	
				6	
				8	
				10	hard drilling, occasional gravels or boulders
				12	
				14	BROWN CLAYEY GRAVEL(GC) medium dense, moist to wet, gravels consist of serpentinite and volcanics (Fill)
			28	16	
				18	
				20	DARK GRAY TO BLACK GRAVELLY CLAY(CL) medium stiff, wet (Fill)
			24	22	
				24	
				26	DARK GRAY TO BLACK SANDY CLAY(CL) medium stiff, wet (Fill)
				28	
				30	strong organic odor
			30	32	
				34	grading to mottled light/dark gray, increase in sand content
			21	36	
				38	DARK GRAY TO BLACK SHEARED SHALE, with serpentinite inclusions, very weak, plastic, highly weathered to soil consistency Seepage encountered at 39 feet
			46	40	
				42	
				44	GRAY SHEARED SHALE, weak, plastic, highly weathered, occasional resistant inclusions of serpentinite
			50/3"		BOTTOM OF BORING 2A @ 45.3 FEET

* Converted to equivalent standard penetration blow counts.

** Approximate ground surface at time of drilling.

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HERZOG ASSOCIATES INC.

Job No: 6636.01.01.1
Appr: CH
Drwn: PD
Date: NOV 1993

LOG OF BORING 2A

UPPER RESERVOIR
St. Helena, California

PLATE 3

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	BROWN SANDY CLAY(CL) medium stiff, moist, with occasional gravels (Fill)
				2	
				4	
				6	BROWN CLAYEY GRAVEL(GC) medium dense, moist (Fill) at 6' harder drilling, possible boulder
				8	
				10	GRAY-BROWN SANDY CLAY(CL) medium stiff to stiff, moist (Fill/Residual)
				12	
				14	GRAY-GREEN SERPENTINITE, extremely closely spaced fractures, weak, friable, highly weathered
			50	16	BOTTOM OF BORING 3A @ 16 FEET No Free Water Encountered

EQUIPMENT: 8" Hollow Auger
LOGGED BY: J. Noble
ELEVATION: 616.5' **
START DATE: 11-17-93
FINISH DATE: 11-17-93

* Converted to equivalent standard penetration blow counts.

** Approximate ground surface at time of drilling.

Huntingdon
Consulting Engineers Environmental Scientists
HERZOG ASSOCIATES INC.

Job No: 6636.01.01.1
Appr: CH
Drwn: PD
Date: NOV 1993

LOG OF BORING 3A

UPPER RESERVOIR

St. Helena, California

PLATE 4

EQUIPMENT: 6" Soil Auger
LOGGED BY: J. Noble

ELEVATION: 617.0' **
START DATE: 11-17-93
FINISH DATE: 11-17-93

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	BROWN SANDY CLAY(CL) medium stiff, moist, with variable gravel content (Fill)
				2	
				4	
				6	
				8	
				10	GRAY CLAYEY SAND(SC) medium dense, moist (Fill)
				12	BROWN SANDY CLAY(CL) medium stiff, moist (Fill)
				14	
				16	DARK GRAY SANDY CLAY(CL) medium stiff, moist, with occasional gravels (Fill)
				18	
			26	20	DARK GRAY SHEARED SHALE, weak, plastic, highly weathered to soil consistency, with yellow streaks
					BOTTOM OF BORING 4A @ 21.5 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Approximate ground surface at time of drilling.

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HERZOG ASSOCIATES INC.

Job No: 6636.01.01.1
Appr: CH
Drwn: PD
Date: NOV 1993

LOG OF BORING 4A

UPPER RESERVOIR

St. Helena, California

PLATE 5

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Blows/ Foot *	DEPTH (FEET)	
				0	BROWN SANDY CLAY(CL) stiff, moist (Fill)
				2	WHITE TO GRAY ANDESITE, closely spaced fractures, moderately hard, moderately strong, highly weathered, very hard drilling at 3.5'
			50/1"	4	grading to red-brown at 4 feet Practical Refusal at 4.5'
					BOTTOM OF BORING 5A @ 4.5 FEET No Free Water Encountered

EQUIPMENT: 6" Solid Auger
LOGGED BY: J. Nol...
ELEVATION: 618.5' **
START DATE: 11-17-93
FINISH DATE: 11-17-93

* Converted to equivalent standard penetration blow counts.

** Approximate ground surface at time of drilling.

Huntingdon
Consulting Engineers Environmental Scientists
HERZOG ASSOCIATES INC.

Job No: 6636.01.01.1
Appr: CH
Drwn: PD
Date: NOV 1993

LOG OF BORING 5A

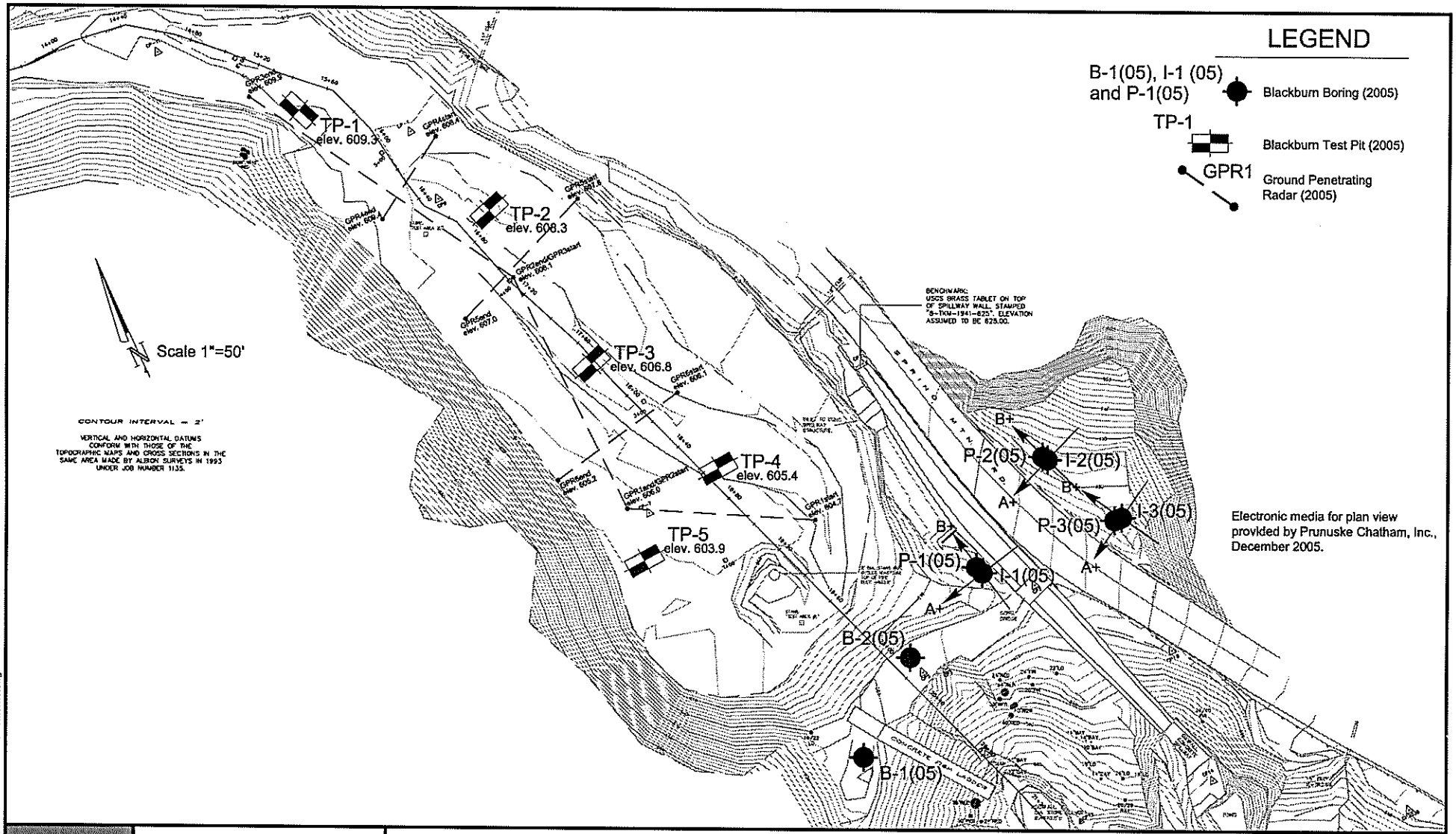
UPPER RESERVOIR

St. Helena, California

PLATE 6

APPENDIX G2

PRELIMINARY BASELINE DATA
(INCLINOMETER AND OBSERVATION WELL)

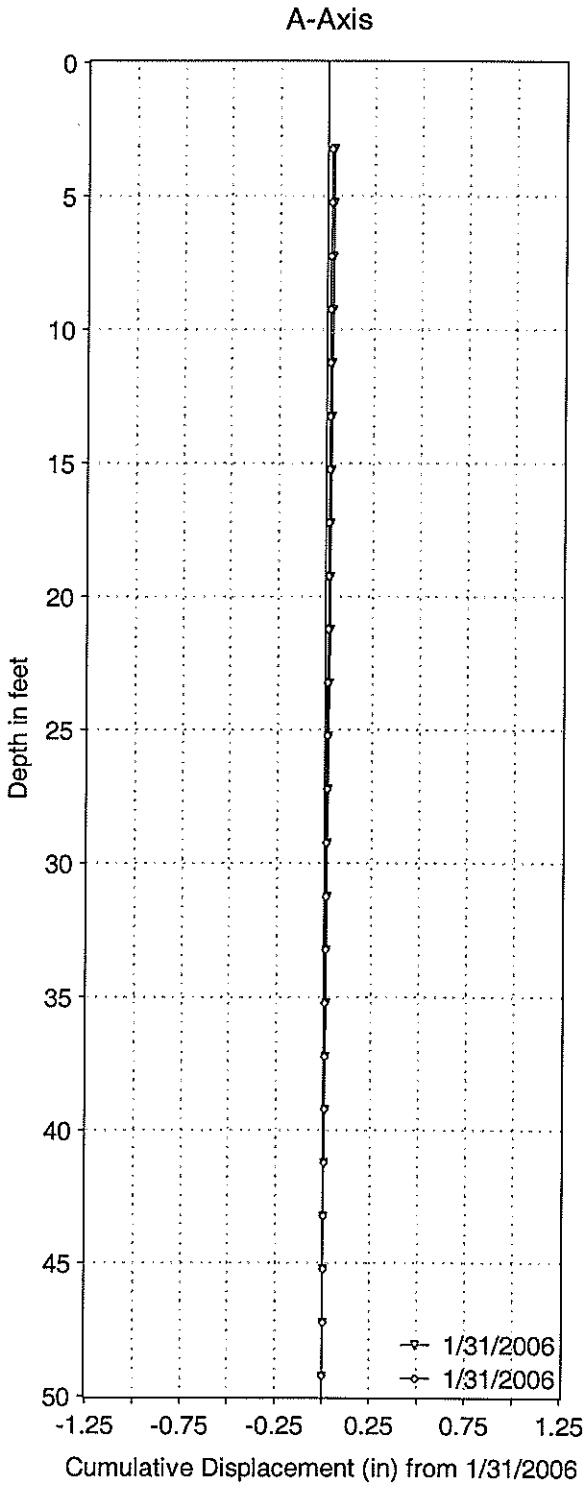


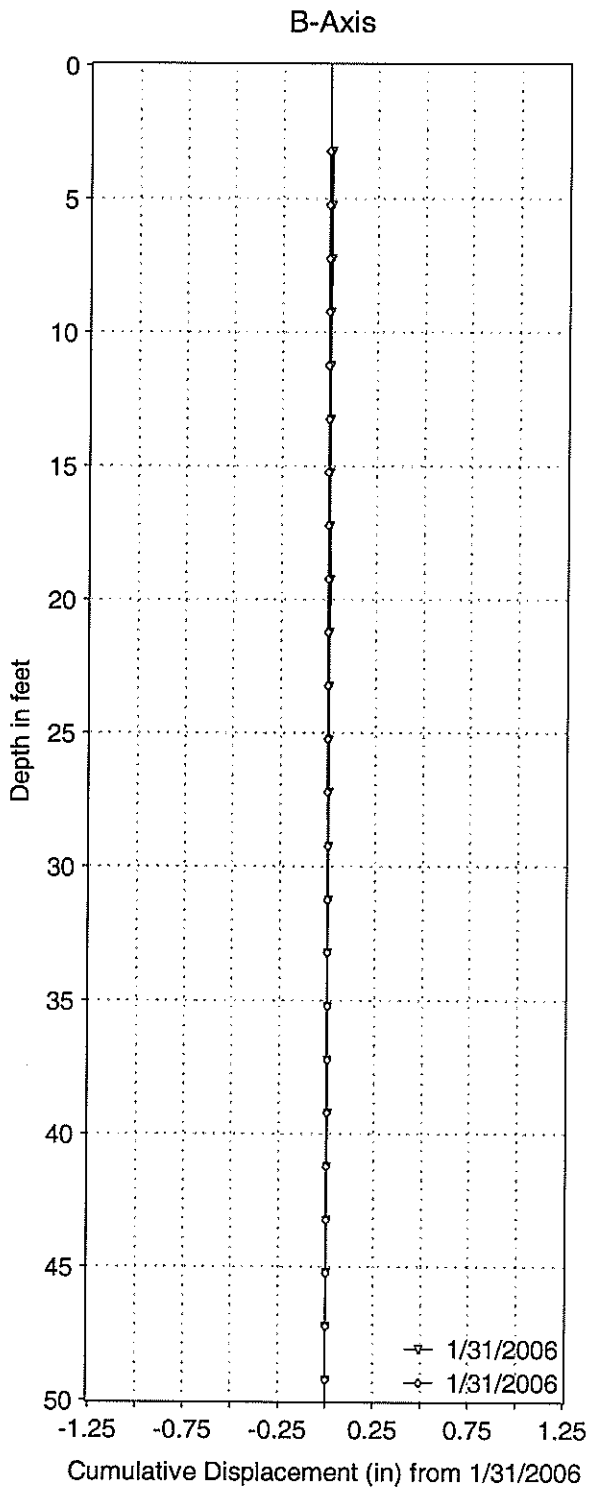
blackburn consulting
11521 Blocker Drive, Suite 110
Auburn, CA 95603
Phone (530) 887-1494
Fax (530) 887-1495
www.blackburnconsulting.com

SITE PLAN/INCLINOMETER ORIENTATIONS
York Creek Dam Removal
St. Helena, California

File 483.1
February 2006
Figure

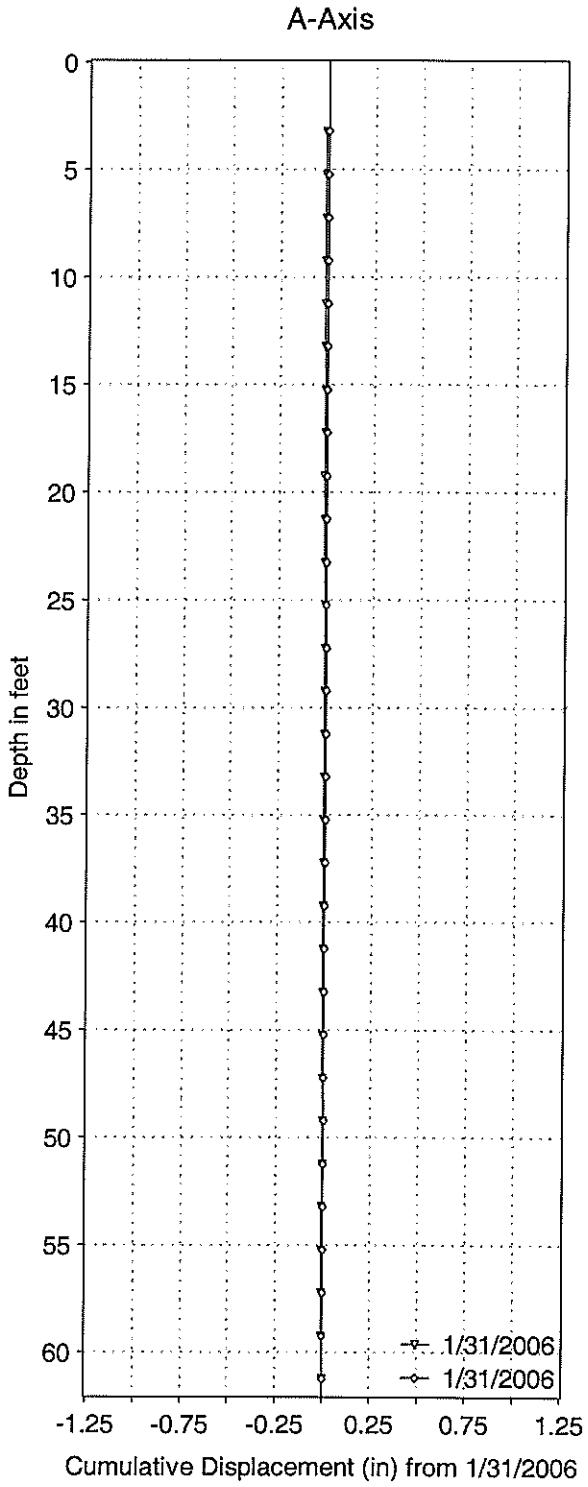
02-14-06 483.1 York Creek Dam Removal Inclinometers.dwg

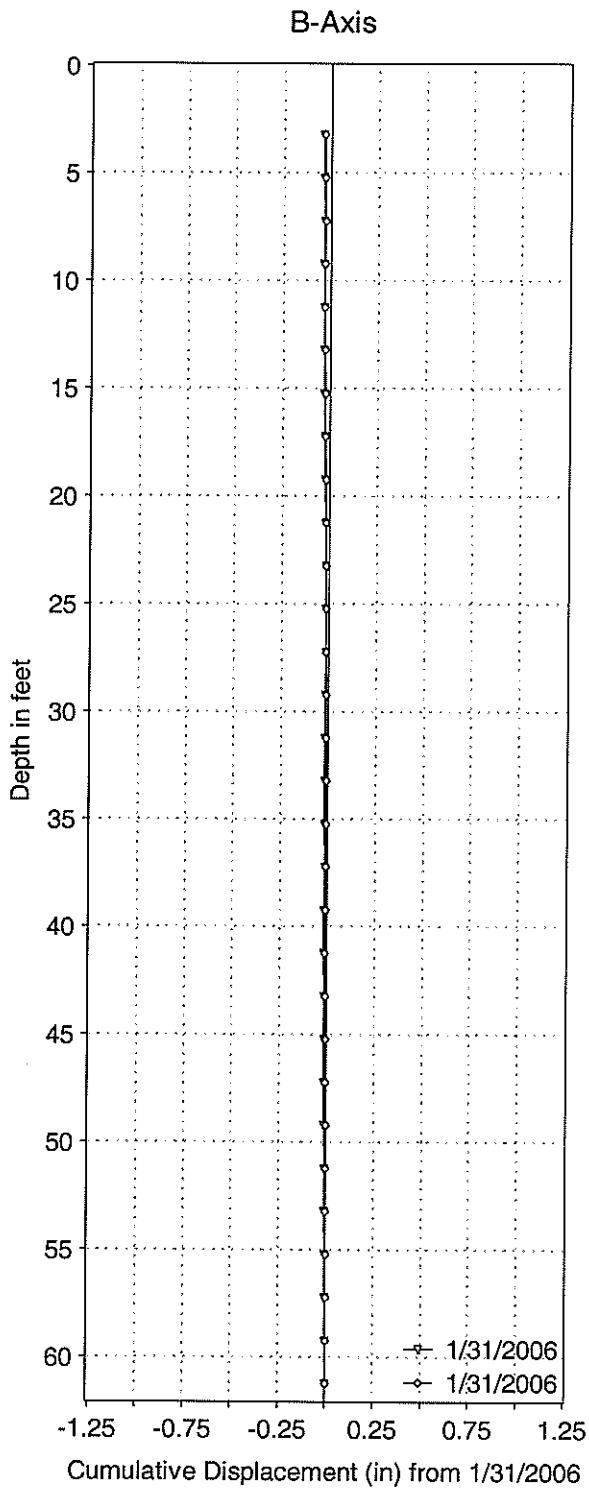




Blackburn Consulting, Inc.
11521 Blocker Drive, Suite 110
Auburn, CA 95603
Job No. 483.1

York Creek Dam
Inclinometer I-1
A+ Orientation S77W





Blackburn Consulting, Inc.
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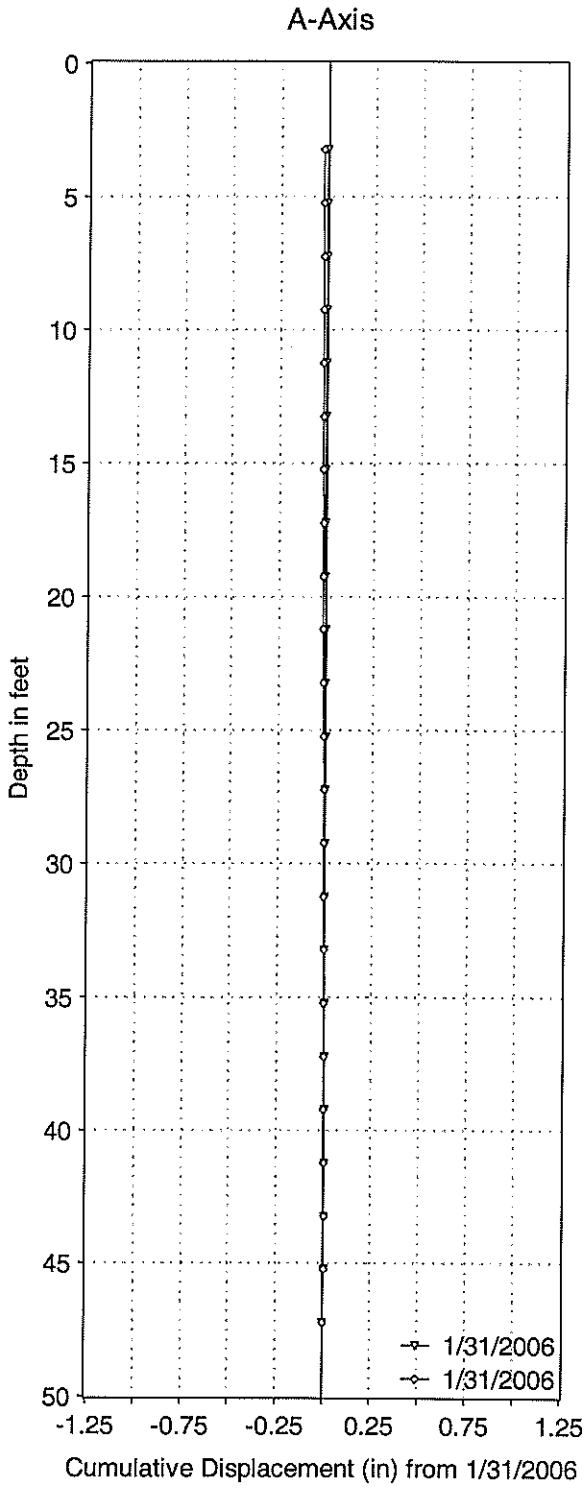
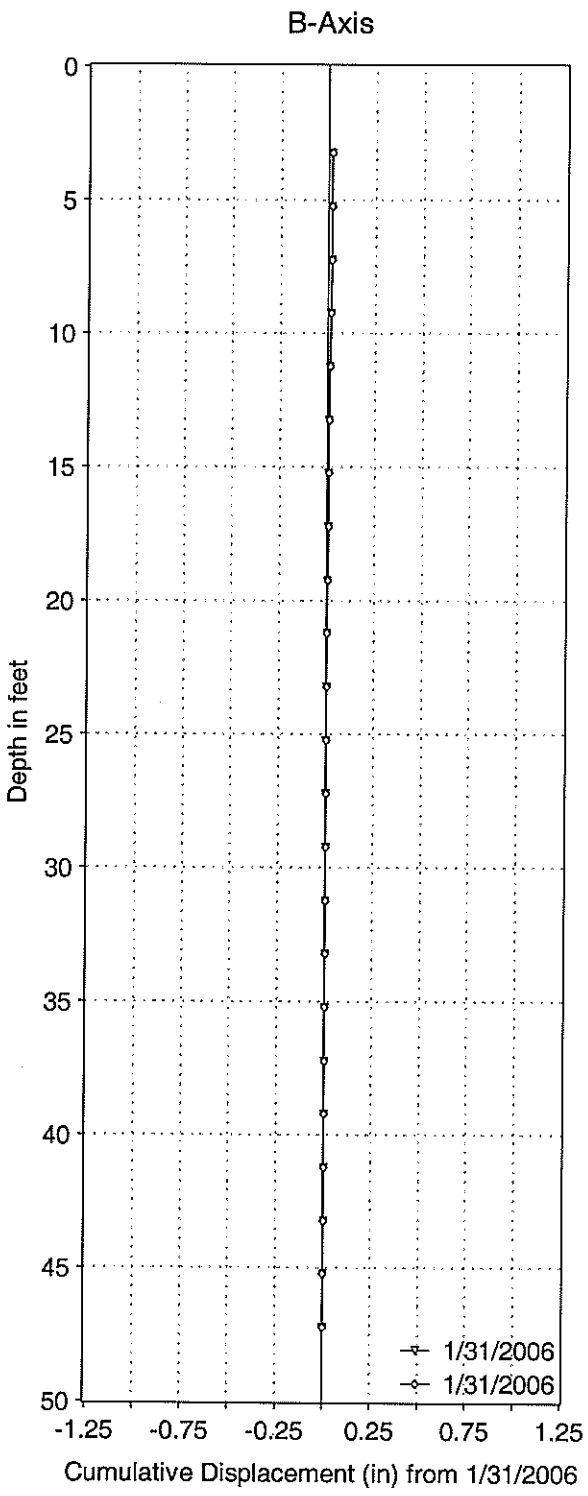
Auburn, CA 95603
Job No. 483.1

York Creek Dam
Inclinometer I-2
A+ Orientation S69W





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Auburn, CA 95603
Job No. 483.1

York Creek Dam
Inclinometer I-3
A+ Orientation S59W

York Creek Dam Removal **483.1**
St. Helena

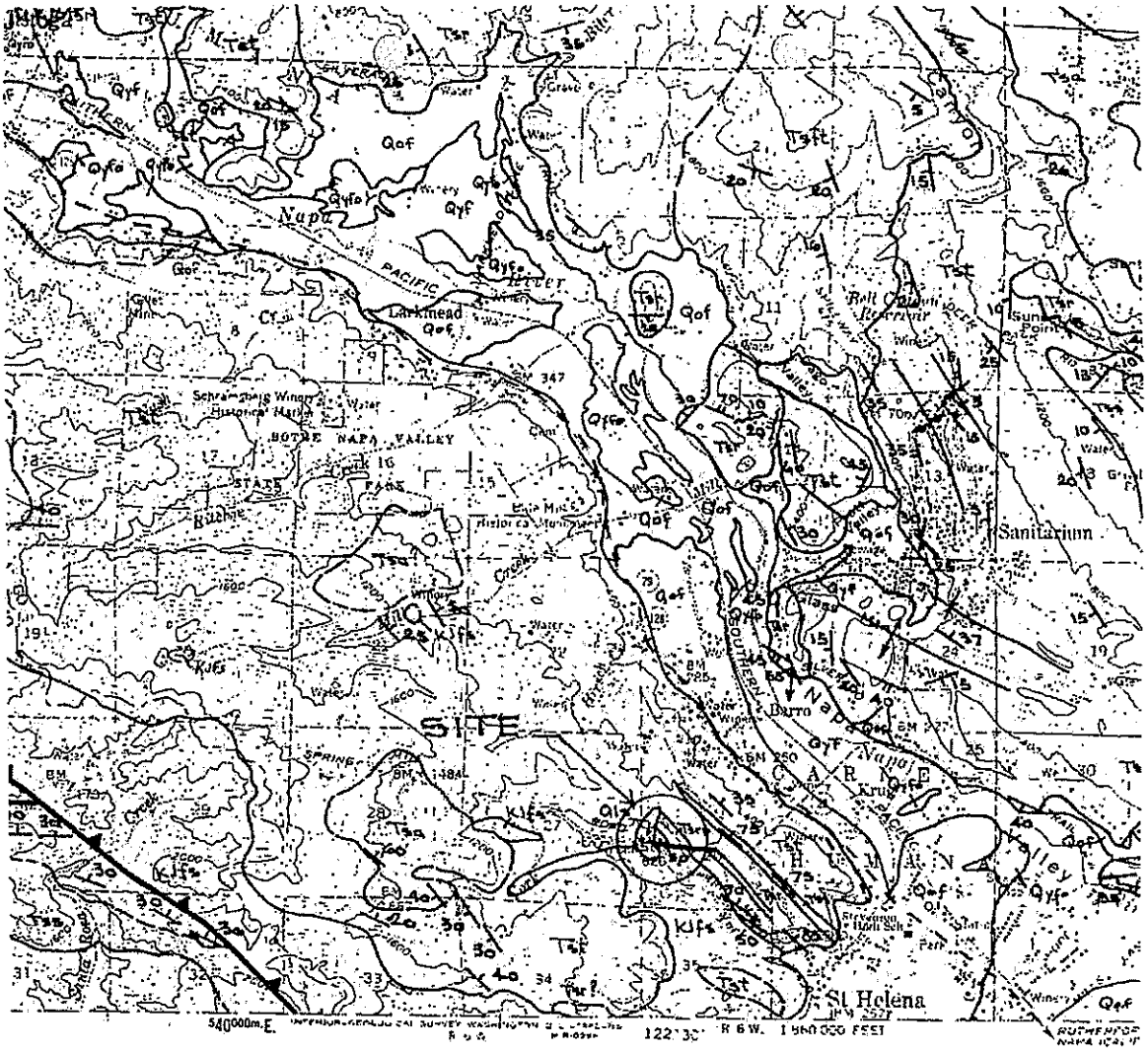
Boring and Pipe Elevations

	Boring Elevation (ft.)	Elevation-top of MW cover	measurement to top of pipe	Elevation at top of pipe
B-1(05)	620.8	621.1	-0.5	620.6
B-2(05)	619.8	n/a	n/a	n/a
P-1(05)	617.1	620.1	-0.9	619.2
P-2(05)	622.4	625.4	-0.31	625.1
P-3(05)	616.7	620.5	-0.62	619.9
I-1(05)	617.2	617.2	-0.23	617.0
I-2(05)	622.4	622.4	-0.23	622.2
I-3(05)	617.0	617.0	-0.23	616.8

Piezometer Groundwater Measurements

Date	11/7/05	11/9/05	1/31/06		
Time	12:00 PM	12:00 PM	11:30 AM		
B-1	26.4	26.1	23.9		
Elev.	594.2	594.5	596.7		
P1(05)	24.2	24.2	19.6		
Elev.	595.0	595.0	599.6		
P2(05)	26.3	25.5	16.9		
Elev.	598.8	599.6	608.2		
P3(05)	22.3	22.4	15.3		
Elev.	597.6	597.5	604.6		

APPENDIX G3
GENERAL GEOLOGY



PRELIMINARY GEOLOGIC MAP of EASTERN SONOMA COUNTY and WESTERN NAPA COUNT

ERN SONOMA COUNTY AND WESTERN

Compiled by

KF
Fox
Jr., J. D. Sims, J. A. Bartow, and E. J. Helley

1973

MF 483
1: 62,500

Qls Landslide
Sp Serpentinite
Tsrp Perlitic rhyolite

Appendix G3. Referenced preliminary geologic map.

APPENDIX G4

QUALITY CONTROL PLAN FOR GEOTECHNICAL PRODUCTS
SITE AND ALTERNATIVES EVALUATION

QUALITY CONTROL PLAN FOR GEOTECHNICAL PRODUCTS

PROJECT AND DOCUMENT INFORMATION

Project Name: Upper York Creek Dam Removal
Document: Site and Alternatives Evaluation
Document Completion Date: 15 March 2005

Project Manager: Karen Berresford

QC Reviewer: David Serafini

Document Author: Michael Ramsbotham

*ITR Reviewer:
*If necessary

The above peer reviewer(s) will perform the Quality Control Review in accordance with SOP EDG-02: "Technical Product Review Policy and Procedures" developed for the Engineering Quality System. The QC reviewer will provide a review of major assumptions, analytical approaches, significant calculations, recommendations and conclusions.

REVIEW CERTIFICATION

PREPARER – I have discussed the above document and review requirements with the QC Reviewer and believe that this review is completed and that the document will meet the requirements of the project.

Preparer's Signature: Michael D Ramsbotham Date: 3/15/05

QC REVIEWER – I have reviewed the assigned Items/Section(s) noted for the above document and believe them to be in accordance with the project requirements, standards of the profession and Corps of Engineers policies and standards.

Reviewer's Signature: _____ Date: 3/15/05

*ITR Chair Signature: NA Date: —

SECTION CHIEF – I have reviewed and resolved all critical and technical issues. I agree that all project requirements, and standards of the profession and Corps of Engineers policies and standards have been met.

Section Chief's Signature: Michael D Ramsbotham For Gomes Date: 3/15/05

_____/s/_____ Date: 12/01/03

Thomas E. Trainer, P.E.
Chief, Engineering Division

Upper York Creek Dam Removal Project
Site and Alternatives Evaluation

15 March 2005

Prepared by the U.S. Army Corps of Engineers
Sacramento District, Sacramento, California

For

San Francisco District, San Francisco, California

1.0 INTRODUCTION

As requested by U.S. Army Corps of Engineers San Francisco District, (SPN), Sacramento District (SPK) undertook an evaluation of the Upper York Creek Dam site. The purpose of the study was to determine the feasibility of proceeding with the plan to remove the Upper York Creek Dam. This is an ecosystem restoration project. The main objective is the reduction / removal of barriers to steelhead trout migration. Upper York Creek Dam is owned and operated by the city of St Helena, CA. The 40-ft high earthen dam is located on York Creek about 1.5 miles north west of downtown St Helena, CA., along Spring Mountain Road. The dam was constructed in 1900. The left abutment spillway was added in 1933. Routine repair and maintenance were performed in the 1970s and in 1985 the dam was modified to bring it into compliance with safety standards. See Attachment 1 Project Location, which is Figure 2 taken from the Historical Resources Evaluation Report.

2.0 SCOPE

SPK's scope of work includes review of existing information and engineering analyses, site visits to evaluate field conditions, new stability analyses, coordination meetings with SPN, attendance at in-progress review meetings, and report preparation.

3.0 GEOLOGY

The dam site is located in the Coast Ranges Geomorphic Province. Based on the "Preliminary Geologic Map of Eastern Sonoma County and Western Napa County", MFS 483, the site is in an area that is geologically complex. Perlitic ryholite (Tsrp), Serpentinite (Sp), sheared shale and sandstone (KJfs), a large landslide (Qls), and a fault are all mapped in the vicinity of the site. See Attachment 2, which is a photocopy of a part of the referenced preliminary geologic map.

At the site, exposures of welded tuff (Sonoma Volcanics) and serpentinite are clearly visible in the materials exposed by the recent landslide.

4.0 SUBSURFACE CONDITIONS / SITE CHARACTERIZATION

The recent exploration performed for the Corps by Treadwell and Rollo indicate the road / pavement section is underlain by fill and then serpentinite at relatively shallow depths. The concrete spillway and left abutment of the dam are also underlain by serpentinite. The serpentinite is generally described on the logs as dark greenish gray to black, moderately to highly weathered, friable to weak, soft to moderately hard, highly fractured

and sheared, with hard cobble to boulder size rock in a softer, weaker more weathered matrix. Core recovery in the serpentinite rock material was poor and in some intervals there was zero recovery. Reported poor recovery at a particular depth may be an indication of a shear zone or potential failure surface that could help define failure plane geometry. Ground water was encountered at about 15 ft below the road surface at B-2 and about 25 ft below the ground surface at B-4.

Fills and backfills, associated with the dam and spillway construction, were also encountered. The fills are variable and consists of sandy silt (ML) / silty sand (SP), clayey sand (SC), silty-clayey sand (SC-SM), gravelly clay (CL), sandy clay (CL), clayey gravel / crushed serpentinite (GP).

The Huntington Supplemental Geotechnical Investigation describes the dam embankment as consisting primarily of soft to medium stiff sandy clays (CL) and clayey sands (SC), with some clayey gravel (GC) layers. In addition the dam fill / backfill is described as erodible and weak. However the Standard Penetration Tests (SPTs) reported on the logs indicate the dam was satisfactorily constructed. The underlying bedrock is described as andesite in faulted contact with sheared shale and serpentinite.

Future exploration or excavation into the dam will likely encounter a heterogeneous fill with lifts / layers of different materials. This variability in subsurface materials will complicate the evaluation of material engineering properties and strength, and subsequent stability analyses.

5.0 DAM SLOPE

It appears from the topographic map that the original dam was constructed with a 2H to 1V upstream slope and a 1.5H to 1V downstream slope, (H = horizontal, V= vertical). The downstream slope appears stable in the areas that have not been affected by the overtopping events. The upstream slope is buried under sediment and other wise obscured by vegetation such that a visual assessment of slope stability was not possible.

6.0 NATURAL SLOPES

The referenced topography was used to estimate native undisturbed slope angles. The right abutment upstream of the dam is standing on slope angles that vary from about 0.8 H to 1.4 H to 1V. The right abutment in the immediate vicinity of the dam is standing at about 0.8 to 0.9 H to 1 V. The abutment slope immediately above the dam is at about a 1.4 H to 1V slope.

The existing highway cut in the tuft is standing at about a 0.4 H to 0.5 H to 1 V slope. The existing highway cut in the serpentine is about a 0.6 to 0.7 H to 1v cut slope.

7.0 WITHOUT PROJECT

Without the proposed project, it is possible that the area would experience a relatively large landslide. The site is underlain by a serpentinite and sheared shale, that is prone to instability and previous geologic mapping indicates the area is a site of an old slide of significant size. There is no way to accurately predict a landslide since hidden defects or existing but hidden planes of weakness often contribute to instability. In addition it is very likely that the failed road cut will continue to exhibit instability and periodic slides particularly in response to above average or intense rainfall. These relatively small landslides will adversely impact the road.

8.0 WITH PROJECT ALTERNATIVES

Alternative 1 involves removing the dam, spillways, outlet works, and all sediment behind the dam.

Alternative 4 involves removing a portion of the dam (notching), the outlet works, and most of the sediment behind the dam. An original stream level fish passage would be constructed using rock weirs.

Alternative 5 involves leaving the dam relatively intact and adding a new fish passage structure up and over the embankment or through the existing left abutment spillway.

9.0 STABILLITY ANALYSES

9.1 General

In general, trying to predict landslides and "back-calculating" strengths needed to evaluate the potential for failure is often difficult. Factors that that influence the analysis and interpretation include: 1) The relative strength of materials in heterogeneous subsurface conditions impact the interpretation of the target material strength, 2) The slip surface analyzed must be the same as the actual rupture surface to effectively back calculate the strength of the deposit, 3) Knowledge of the pore water pressure in required to determine effective stresses and therefore strength, 4) All slides have a three dimensional component, 5) Progressive failure in strain softening materials will also effect interpretation of strength, 6) Strength is defined by a friction angle and cohesion and determination of these parameters individually is typically not possible without significant data.

When designing levees and dams, Corps criteria typically requires the minimum factor of safety for long term steady state conditions to be 1.5 and for short term or end of construction conditions, the minimum required factor of safety is typically 1.3. For this project, circumstances indicate that acceptable minimum factors of safety could be lower.

The Project Development Team should evaluate the risks and adopt a level of conservatism (factors of safety) appropriate for the project.

9.2 DWR Stability Modeling

The California Department of Water Resources ran numerous stability analyses in their evaluation. Because of the difficulties and unknowns involved with the analyses, the results should be viewed as qualitative and not a direct predictor of the potential for sliding to occur. Most importantly, since their evaluation assumed dry abutment conditions, which is applicable only part of the time, the calculated factors of safety are optimistic. Nonetheless, their analyses are helpful in determining what might be the best course of action and selecting an alternative.

9.2.1 Stability Analyses Summary

A summary of DWR's work is presented below and in Table 1 Results of DWR Stability Modeling.

Referencing DWR Figures A1 and A2 – The postulated failure surface in this model comes under the road at a shallow depth and is altered by the spillway. Only the upper part of the dam near the left abutment is engaged by the slide. Cohesion appears to be the largest contributor to stability for this slope and slide geometry. Removal of the dam reduces the calculated Factor of Safety by 12% to 15%.

Referencing DWR Figures A3 and A4 – The postulated failure surface for this model passes under the road and spillway before encountering the dam. Again, only the upper part of the dam near the left abutment is engaged by the slide. Friction is the largest contributor to sliding resistance for this slope and slide geometry. Removal of the dam reduces the calculated Factor of Safety by 8.5% to 13.5%.

Referencing DWR Figures A5 and A6 – The postulated failure surface is similar to the A3/A4 model but the sliding mass is larger with a failure surface extending to a higher elevation on the hillside. Removal of the dam reduces the calculated Factor of Safety by 6% to 10%.

Referencing DWR Figure A7 - The postulated failure surface is similar to the A6 model but the sliding mass is even larger with a failure surface extending to a much higher elevation on the hillside. This model produced some of the lowest calculated factors of safety.

Referencing DWR Figures A8 and A9 – These models simulate an abutment slide with postulated failure surfaces that initiate on the road, come under the spillway, and toe our at the bottom of the bank at the creek level after the dam has been removed. Even with relatively low selected strengths (c = 800 and phi = 8.5 to c = 950 and phi = 0), the calculated factors of safety indicate the new cut slope would be stable at a slope angle of

roughly 1.5H to 1V. However additional stability analyses should be run including groundwater effects.

Referencing DWR Figure A10 – This model is similar to A6 but the postulated failure mass is larger and the failure surface deeper.

Referencing DWR Figure A11 - This model is very similar to A7. The postulated failure mass is similar in size and the postulated failure surfaces are nearly coincident. Thus calculated factors of safety for model A11 are similar to A7 calculated factors of safety and are also relatively low.

9.2.2 Conclusions:

Base on review of DWR's work it appears that,

The smaller slides that form in the over-steepened area where slides have already occurred are more likely to occur than the relatively large scale slides encompassing the entire hillside.

The bigger the slide mass and the deeper the slide plane the less overall influence the dam contributes to the stability of the slide.

The creek bank or dam abutments are likely to be stable after removal of the dam.

9.3 Corps of Engineers Sacramento District (SPK) Stability Modeling

SPK performed two-dimensional stability modeling utilizing Utexas4 and the cross section data provided for York Creek Station 20+20. The section is presented on the Prunuske Chatham Inc. York Creek Dam Cross Sections drawing, Sheet 9 of 11. See Attachment 3. Over 400 separate analyses were run, of which only 10 models were selected and are presented with the report. See Figures C-1 through C-4. The major difference in our models compared to DWR's work is that the SPK modeling assumed groundwater is present and that the groundwater elevation decreased with the removal of the dam. (Groundwater is indicated on the exploration logs.)

9.31 Determination of Strengths

The Treadwell & Rollo work included strength testing of serpentinite cores obtained from the borings. Overall though, core recovery was poor and is some intervals there was no recovery. Since only some of the better rock was retrieved and tested, the strength data is somewhat skewed. Thus there is uncertainty in the selection of shear strength based on the laboratory test data.

The reported unconfined compressive strengths and direct shear strength are relatively high. The unconfined compressive strengths range from roughly 1,100 psf to 11,100 psf, (psf = pounds per square foot) For the direct shear tests the maximum, minimum, and average calculated cohesions are 2,754 psf, 761 psf, and 2,093 psf, respectively and the maximum, minimum and average calculated friction angles are roughly 42°, 25.5°, and 37°, respectively. All the individual direct shear data points were plotted together and based on this composite plot a "conservative" estimate of the mass strength of the serpentinite can be represented by a cohesion of 1000 psf and a friction angle of 37°.

Starting with an estimated reconstructed geometry for the existing slide and assumed coincident high groundwater a "back calculated" strength for a Factor of Safety of about 1 was determined. This effort turned out to be more trouble than expected, and is probably a reflection on the complex surface and selected slide plane geometry. After numerous stability runs, a strength with cohesion = 600 psf and friction angle =18° was selected to represent the "residual" strength along a hypothetical failure plane. The existing near vertical head scarp is marginally stable and probably is representative of an induced tension fracture. See Figure C-1.

Thus selected "residual" and "conservative" strengths were used in our modeling to help predict the impacts, removing the dam, will have on the road and resulting creek bank cut slopes.

9.3.2 Stability Analyses Summary

A summary of SPK's stability analyses is presented below and in Table 2 Results of SPK Stability Modeling.

Referencing Corps Figures C-2 through C-4 – Using the relatively low selected "residual" strength, a 1H to 1V cut slope in the serpentinite bedrock was determined to be stable. The calculated Factor of Safety with a high and low ground water level was determined to be 1.2 and 1.3, respectively. When the strength was increased to the "conservative" value, the calculated Factor of Safety increased to 2.2. The near vertical headwall portion of the calculated failure surface represents an induced tension crack.

Referencing Corps Figures C-5 through C-7 – In this model, a planar failure surface / geometry was selected and input. The failure surface was selected based on engineering judgment. Using the relatively low selected "residual" strength, the calculated Factor of Safety with a high and low ground water level was determined to be 1.1 and 1.2, respectively. The selected failure surface does not represent the surface with the lowest possible factor of safety for this type of slide geometry, but it is an indicator of how the existing slide scar and road cut may behave with the dam removed. Since a portion of the driving mass has been removed due to previous landslides, the current slope / geometry has increased stability. When the strength was increased to the "conservative" value, the mass appears more stable and the calculated Factor of Safety increased to 2.2. The near vertical headwall portion of the calculated failure surface represents an induced tension crack.

Referencing Corps Figures C-8 through C-10 – In this model, the computer program was allowed to search for a circular surface with a minimum factor of safety. Using the relatively low selected "residual" strength, the calculated Factor of Safety with a high and low ground water level was determined to be 0.9 for each case. Because of the size and depth of the sliding mass involved, the difference in the high and low groundwater level was immaterial. When the strength was increased to the "conservative" value, the mass appears more stable and the calculated Factor of Safety increased to 1.8.

These calculated large circular failure surface are theoretical and do not appear to be representative of potential failure surfaces in this geologic setting. The results are used to shape overall engineering judgment as to the likelihood that dam removal will instigate a slide. In addition, for a slide mass as large as depicted in figures C-8 through C-10, the relative contribution of the existing dam to stability is minor because the size of the slide mass is much greater than the mass of the dam.

9.3.3 Conclusions:

Base on the Corps' modeling, it appears that,

Abutment stability will likely be controlled by a hidden defect or plane of weakness.

The creek bank or dam abutments will most likely be stable after removal of the dam.

It is unlikely that subsurface conditions will become so well know that the stability of the roadway and abutments can be 100 percent assured.

The bigger the slide mass and the deeper the slide plane the less overall influence the dam contributes to the stability of the slide.

Groundwater has a significant influence on stability.

Designs should include measures to control and measure groundwater levels at the site.

Two dimensional stability analyses should be viewed as a tool and not the definitive answer.

There is uncertainty in the selection of shear strength for the serpentinite based solely on the laboratory test data. Overall, core recovery in the sepentinite was poor and is some intervals there was no recovery. Since only some of the better rock was retrieved and tested, the strength data may be skewed.

Engineering judgment is crucial in the evaluation of this complex three-dimensional problem. (Good geotechnical engineering always starts with an understanding of site geology.)

10.0 SUMMARY & OVERALL CONCLUSIONS

Based on the information reviewed, observation of the site and experience, it appears that the removal of all or a portion of the existing embankment is technically feasible.

Removal of all or a portion of the existing dam is expected to have no adverse impact on the road or left abutment concrete spillway.

The spillway structure should not be removed. The spillway should be left in-place and backfilled to provide continued support for the road.

Landslides from above the road will continue to adversely impact the road.

At this point the geotechnically preferred alternative is to partially remove or notch the existing dam, locating the notch as far toward the right bank as possible. Cut slopes on the order of 1.5H to 1.0V are reasonable. Required excavations may result in localized unstable areas. (By making the minimum excavation required to achieve an adequate fish passage, there is less potential impact and thus less chance the abutment will be adversely affected.)

Erosion protection should be included at the toe of the new cut slopes in the vicinity of the dam.

Additional explorations and stability analyses are highly recommended for final design. The potential for both large scale and small abutment slides should be further evaluated in the final design phase.

It is unlikely that subsurface conditions will become so well know that the stability of the abutments can be 100 percent assured. Therefore an adaptive management approach should be implemented. Monitor the site before, during, and after the construction of the selected alternative

11.0 RECOMMENDATIONS

11.1 Explorations

The existing exploration data should be reviewed in detail during final design.

New explorations should be scheduled to gather additional information about the dam materials and the abutments. Estimates of dam and abutment material strengths will be needed to further evaluate cut slope stability. Add at least 1 exploration on the left abutment, about 30 ft left of the baseline as shown on the Prunuske Chatam Inc. drawings and 1 exploration on the right abutment about 40 ft. right of the baseline. Both explorations should terminate about elevation 570 ft. Emphasis should be on gathering

strength data on the existing dam materials and the bedrock that will form the abutments / banks of the new channel.

11.2 Instrumentation

Install at least 3 inclinometers to monitor for slope movement and slide plane development in the vicinity of the existing dam and slide. Install at least 3 piezometers to monitor groundwater levels in the same are being monitored by the inclinometers. Install several surface survey monuments along the top of both spillway wall to monitor horizontal and vertical movements. Establish at least two stable temporary benchmarks.

Consider installing vibrating wire piezometer transducers to measure pore pressures and coaxial Time Domain Reflectometry (TDR) cable to monitor slope movement.

11.3 Future Analyses

Future stability analyses should focus on evaluating the stability of both abutments cut slopes formed by the removal of the dam.

11.4 Design

Include a dewatering trench / french drain along the east side of the road in the vicinity of the current slide scar.

Consider drilling horizontal drain through the weep holes in the bottom of the existing spillway before backfilling the spillway.

Include a granular / gravel drainage layer at the bottom of the spillway backfill.

Provide for a flexible design that can be adapted to differing site conditions and changes of abutment geometry.

Consider installing horizontal drains in the left abutment / cut slope to aid in dewatering the slope after the dam is removed.

11.5 Construction

Include provision to instrument and monitor the hillside, road and spillway during removal of portions of the dam.

Have a geotechnical engineer or engineering geologist on site to evaluate encountered conditions and modify the excavation as needed to provide a stable slope and fit the pre - dam natural creek bank.

11.6 Adaptive Management

Acknowledge that abutment stability cannot be 100 percent assured because of the possibility of hidden unknown defects will still exist after final design is complete and commit to protecting the roadway.

After construction, continue to monitor the site until it can be reasonably concluded dam removal has not impacted the road. The frequency of instrumentation readings and visual inspection should be relatively high at first and may decrease over time, as long as a problem has not developed. In unusually wet years, monitoring should increase and after periods of prolonged heavy rain the site should be inspected and instruments read.

Michael D. Ramsbotham, P.E., G.E.
Soil Design Section

Vinh Tran
Dam Safety & Infrastructure Support Section

REFERENCES:

Map of Topography with Cross Sections, York Creek Dam and Reservoir, City of St Helena, Napa County, California prepared by Prunuske Chatham, Inc. Scale 1 inch to 20 ft., dated 2005.

"Preliminary Geologic Map of Eastern Sonoma County and Western Napa County", Miscellaneous Field Studies Map MF483 prepared by Fox, Sims, Bartow, and Helley, published 1973.

California Department of Water Resources (DWR) Report titled York Creek Dam Removal – Slope Stability Analysis.

Field and Laboratory Activities Report, Upper York Creek Dam Removal Project, Napa County California prepared by Treadwell & Rollo Inc., dated 20 April 2004, Project 3772.01

Historical Resources Evaluation Report For the Proposed Removal of an Earthen Dam and Diversion Structure on York Creek Near the City of St. Helena in Napa County, California, prepared for the City of St. Helena by the Bright Eastman, Anthropological Studies Center, Sonoma State University, Rohnert Park, CA dated February 2003.

Report, Supplemental Geotechnical Investigation, Upper Reservoir Dam, St. Helena, California, prepared by Huntington Consulting Engineers and Environmental Scientist and Herzog Associates, Inc. dated November 29, 1993.

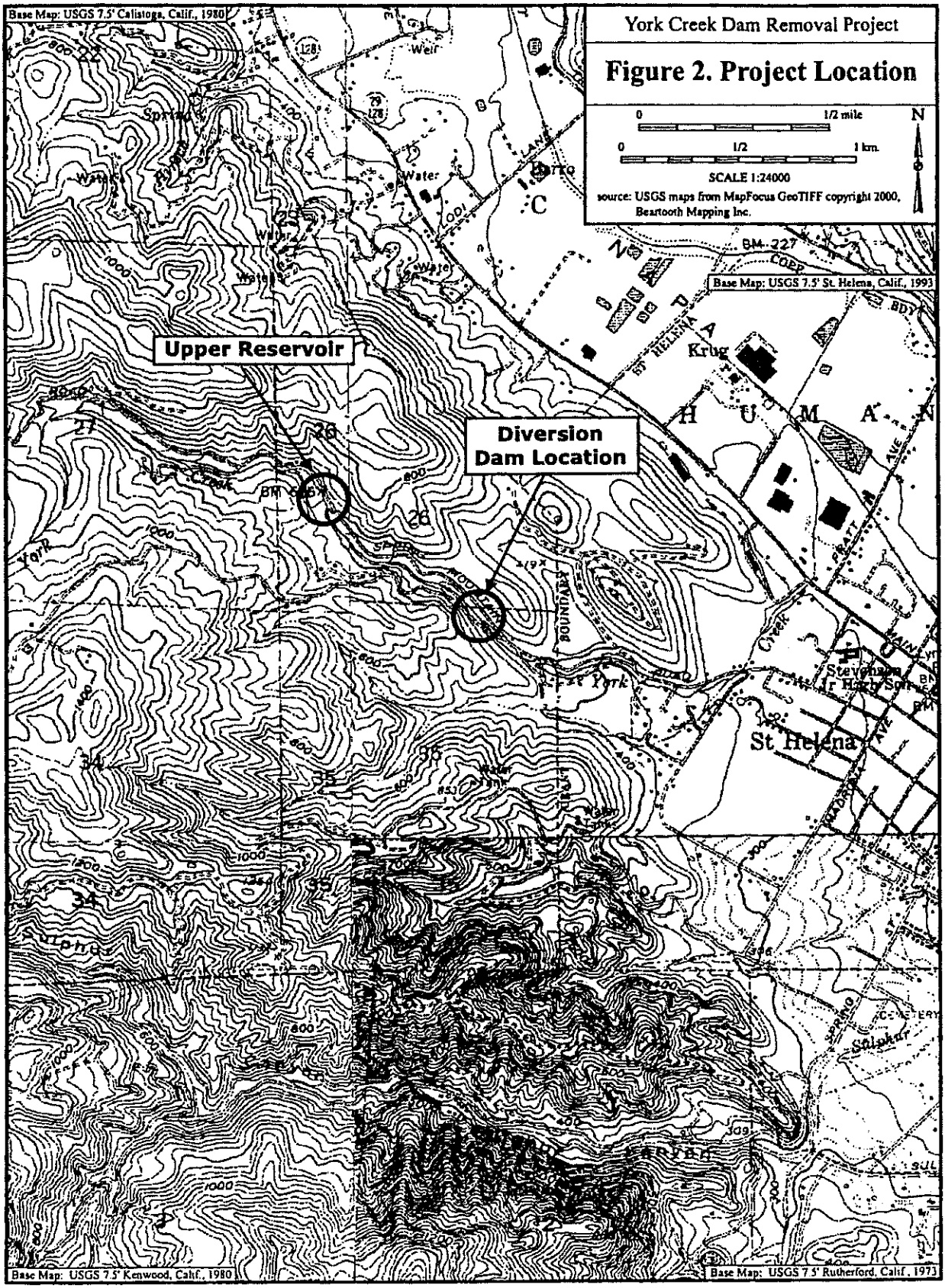
Progress Report (Draft) Upper Reservoir Dam, St. Helena, California, prepared by Huntington Consulting Engineers and Environmental Scientist and Herzog Associates, Inc. dated August 24, 1993.

Table 1. Results of DWR Stability Modeling

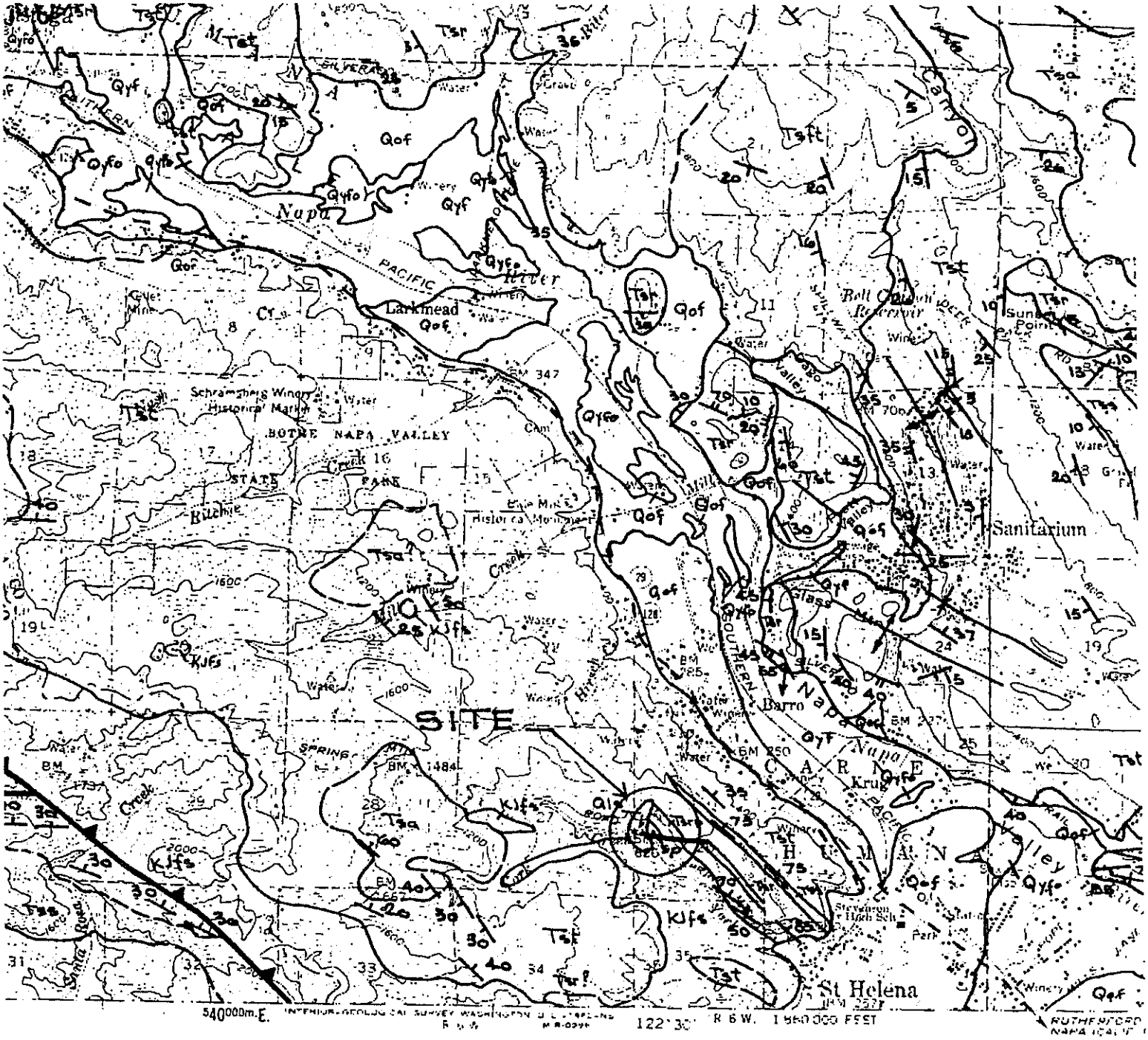
Model Number	Calculated Factor of Safety	Description
A1	1.9 – 2.3	Small east hillside slide with dam.
A2	1.7 – 2.0	Small east hillside slide without dam.
A3	1.6 – 1.9	Small east hillside slide with dam.
A4	1.4 – 1.7	Small east hillside slide without dam.
A5	1.2 – 1.8	Slightly larger east hillside slide with dam.
A6	1.1 – 1.7	Slightly larger east hillside slide without dam. Similar to A10.
A7	0.7 –1.6	Largest east hillside slide without dam. Similar to A11.
A8	2.0 – 2.2	Small left abutment slide after dam removal.
A9	2.0 – 2.2	Small left abutment slide after dam removal.
A10	1.1 –1.7	Slightly larger east hillside slide without dam. Similar to A6.
A11	0.6 – 1.4	Largest east hillside slide without dam. Similar to A7

Table 2. Results of SPK Stability Modeling

Model Number	Calculated Factor of Safety	Assumed Strength	Description
C1	1.0	600 psf and 18 degrees	Existing slide "back calculated" strength with Factor of Safety set to 1
C2	1.3	600 psf and 18 degrees	Left Abutment without dam and low water table
C3	1.2	600 psf and 18 degrees	Left Abutment without dam and high water table
C4	2.2	1000 psf and 37 degrees	Left Abutment without dam and high water table and conservative strength
C5	1.2	600 psf and 18 degrees	Large Slide without dam and low water table
C6	1.1	600 psf and 18 degrees	Large Slide without dam and high water table
C7	2.2	1000 psf and 37 degrees	Large Slide without dam and high water table and conservative strength
C8	0.9	600 psf and 18 degrees	Large Circular Slide without dam and low water table
C9	0.9	600 psf and 18 degrees	Large Circular Slide without dam and high water table
C10	1.8	1000 psf and 37 degrees	Large Circular Slide without dam and high water table and conservative strength



Attachment (1)



PRELIMINARY GEOLOGIC MAP of EASTERN SONOMA COUNTY and WESTERN NAPA COUNTY

ERN SONOMA COUNTY AND WESTERN

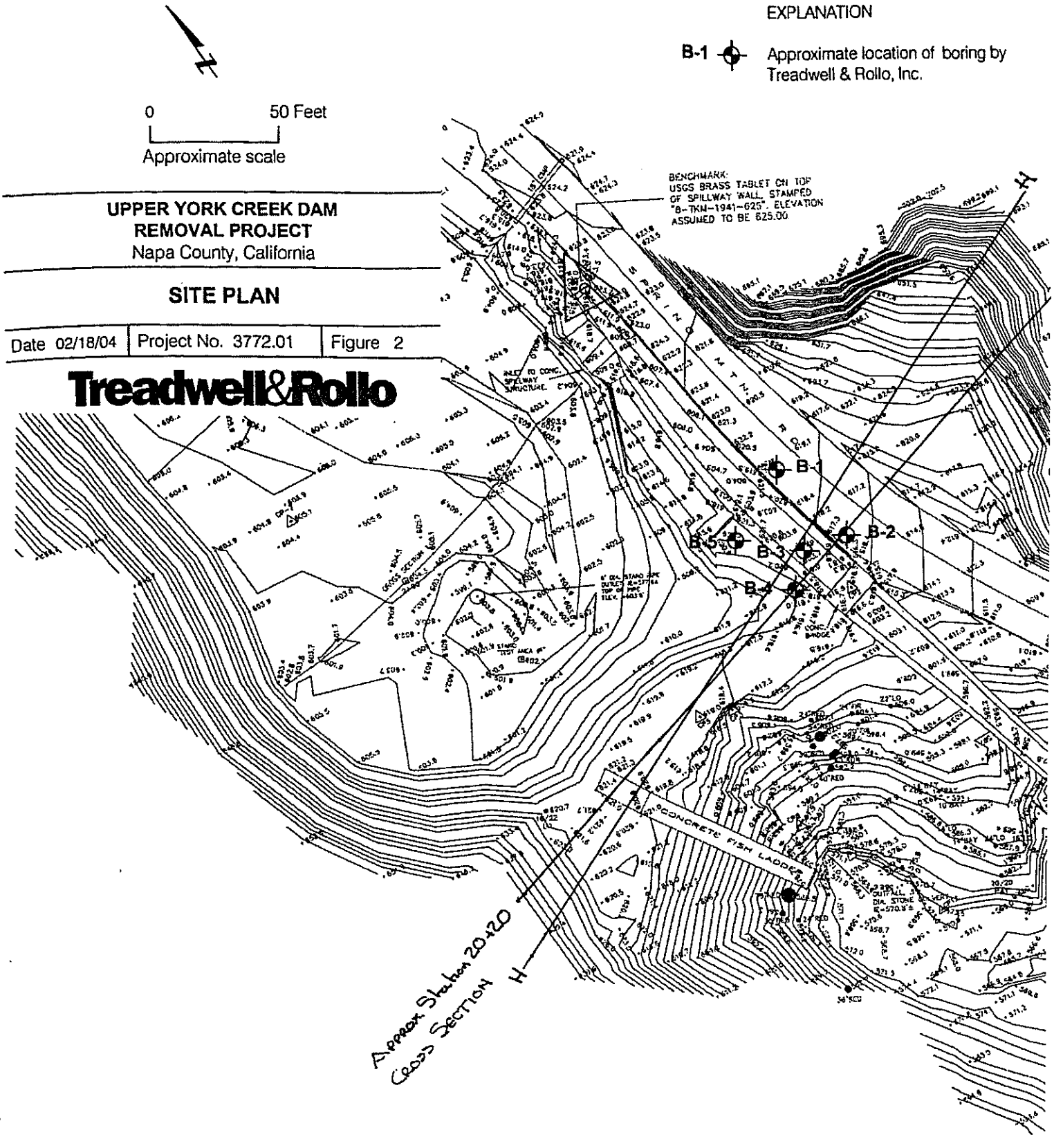
Compiled by

KF
Fox
Jr., J. D. Sims, J. A. Bartow, and E. J. Helley

1973

MF 483
1:62,500

Qls Landslide
Se Serpentinite
Tsro Perlitic rhyolite



UPPER YORK CREEK DAM REMOVAL PROJECT
Napa County, California

SITE PLAN

Date 02/18/04	Project No. 3772.01	Figure 2

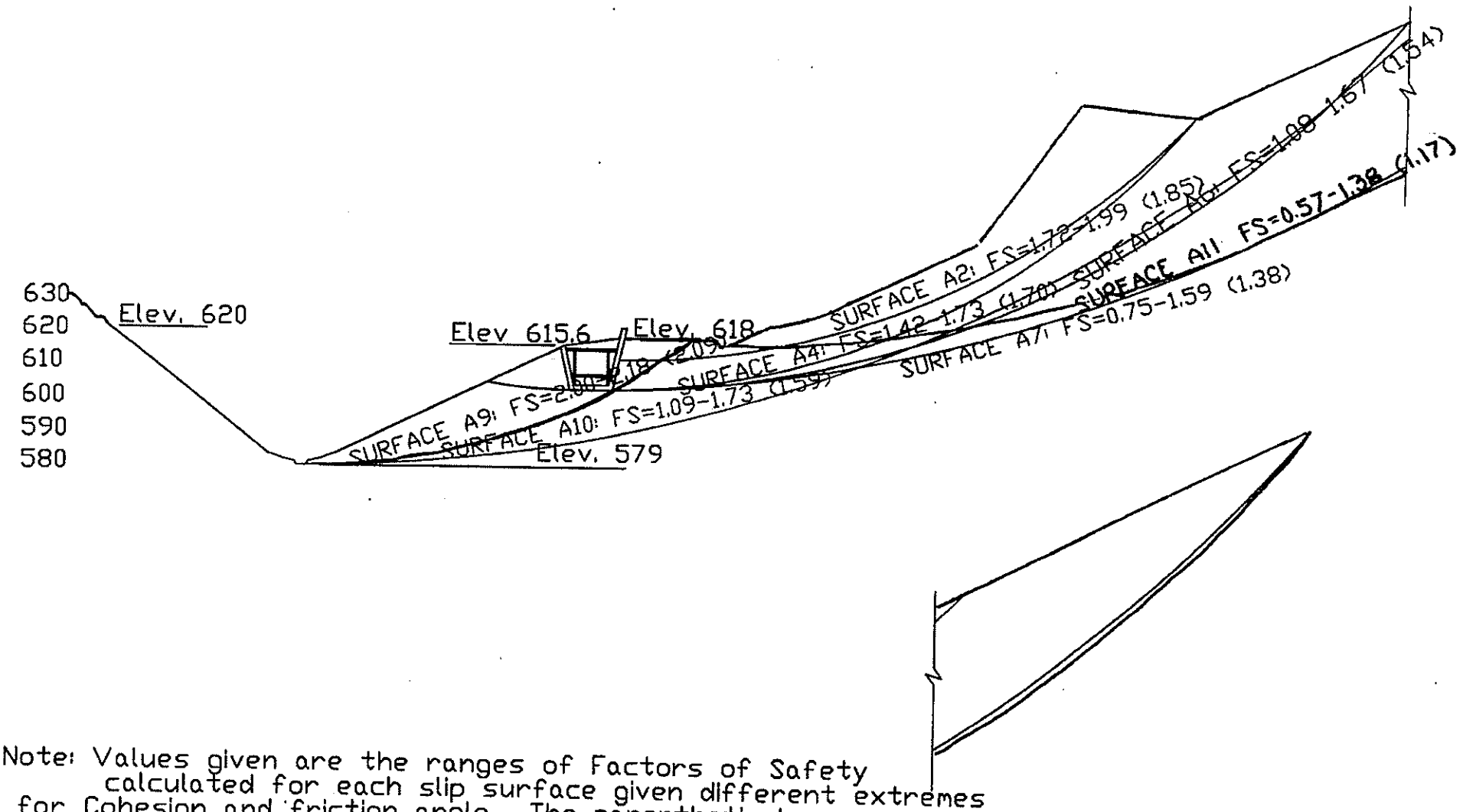
Treadwell&Rollo

377201_SitePlan.dwg

Reference: Base Map provided by US Army Corp Titled "Map of Topography with Cross Sections, York Creek Dam & Reservoir, City of St. Helena, Napa County, California,", dated March 2002, prepared by Albion Surveys, Inc. of St. Helena, California. (USACOE File No. 1135.02-2TP.dwg). Boring locations surveyed by Winzler and Kelly Consulting Engineers of Santa Rosa, California on 23 October 2003 using USGS Brass Tablet on top of spillway as Benchmark (stamped B-TKM-1941-625').

Attachment (3)

FIGURE A - CROSS SECTION H-H



Note: Values given are the ranges of Factors of Safety calculated for each slip surface given different extremes for Cohesion and friction angle. The parenthetical value corresponds to a Cohesion value of 300 psf and a friction angle of 26.0 degrees.

FIGURE A1 - CROSS SECTION H-H

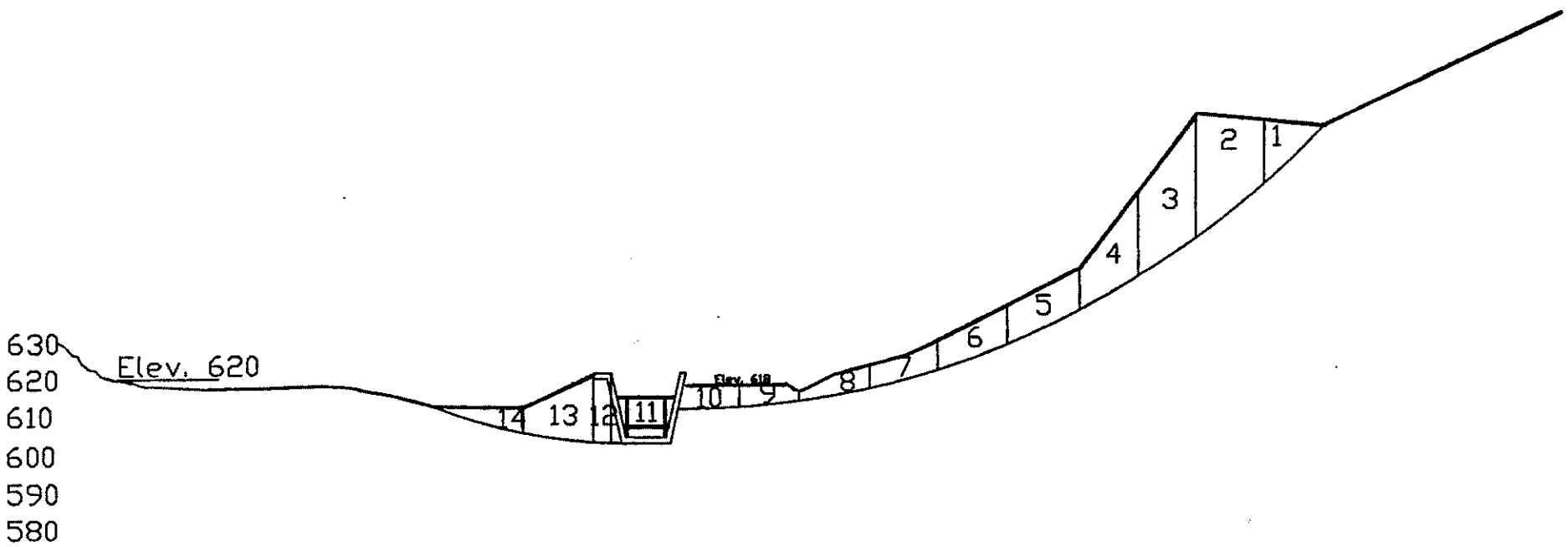


FIGURE A2 - CROSS SECTION H-H

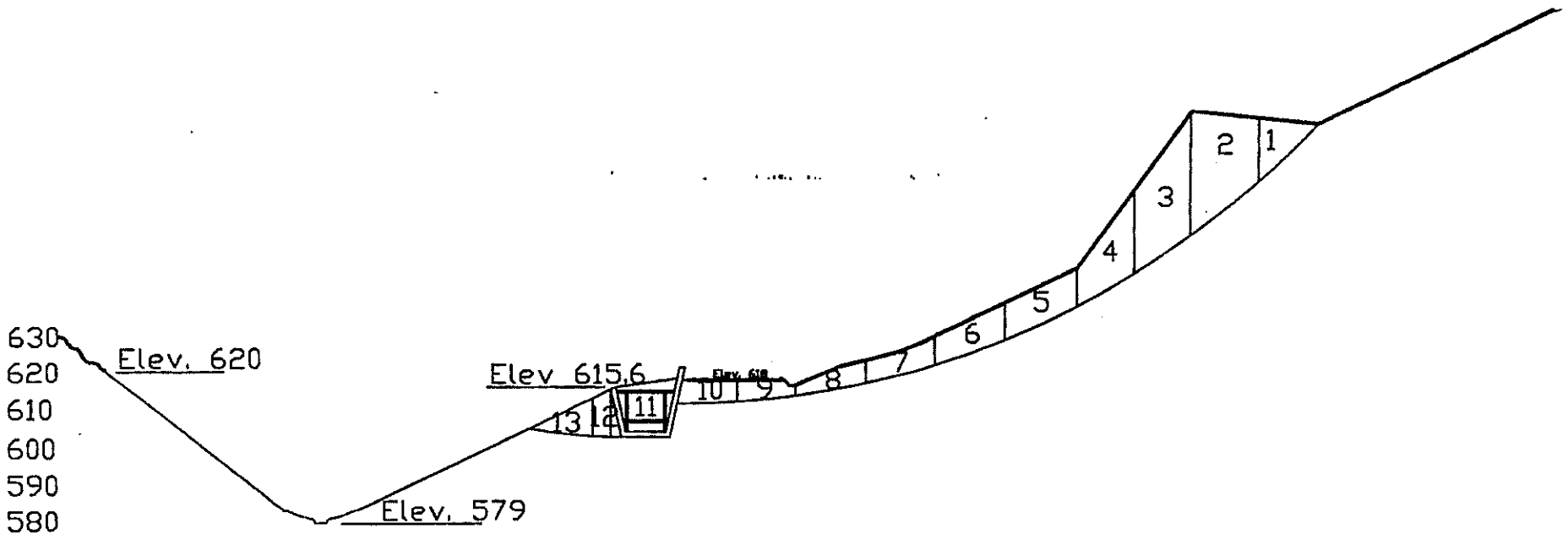


FIGURE A3 - CROSS SECTION H-H

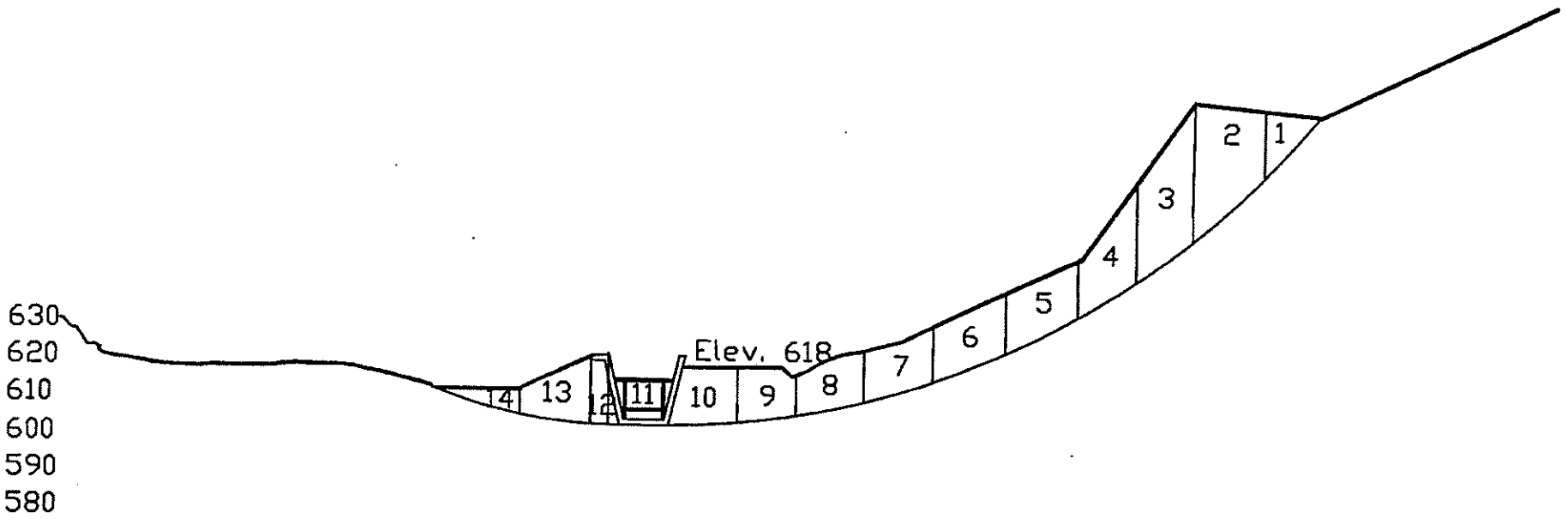


FIGURE A4 - CROSS SECTION H-H

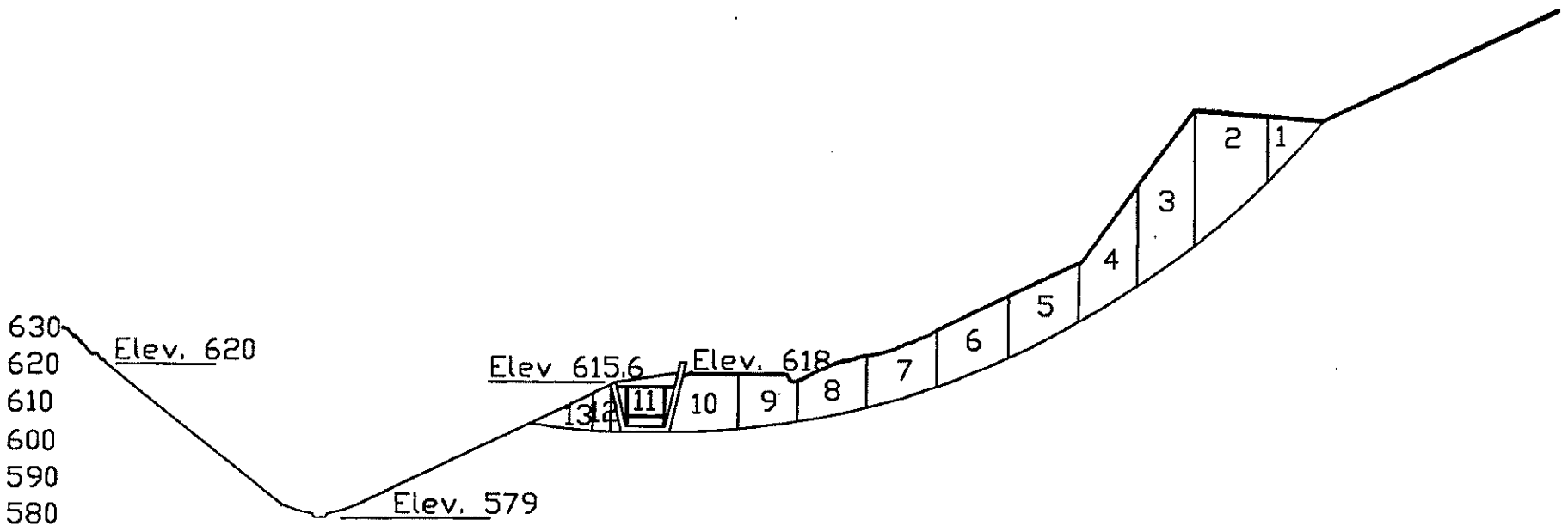


FIGURE A5 - CROSS SECTION H-H

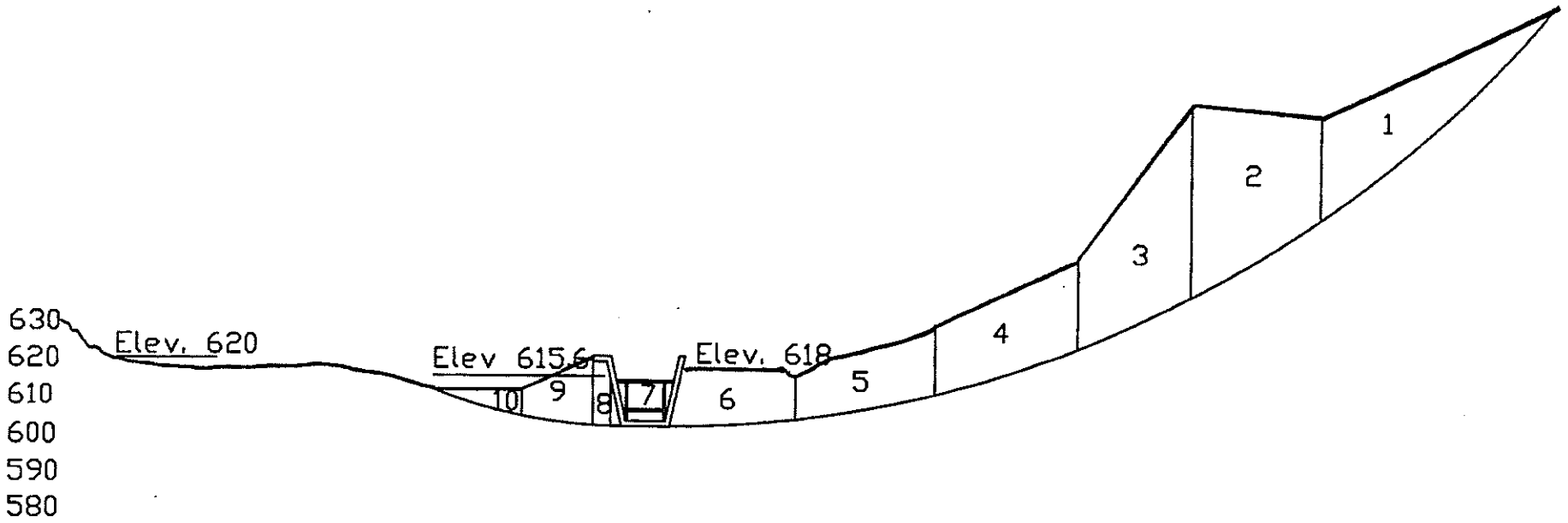


FIGURE A6 - CROSS SECTION H-H

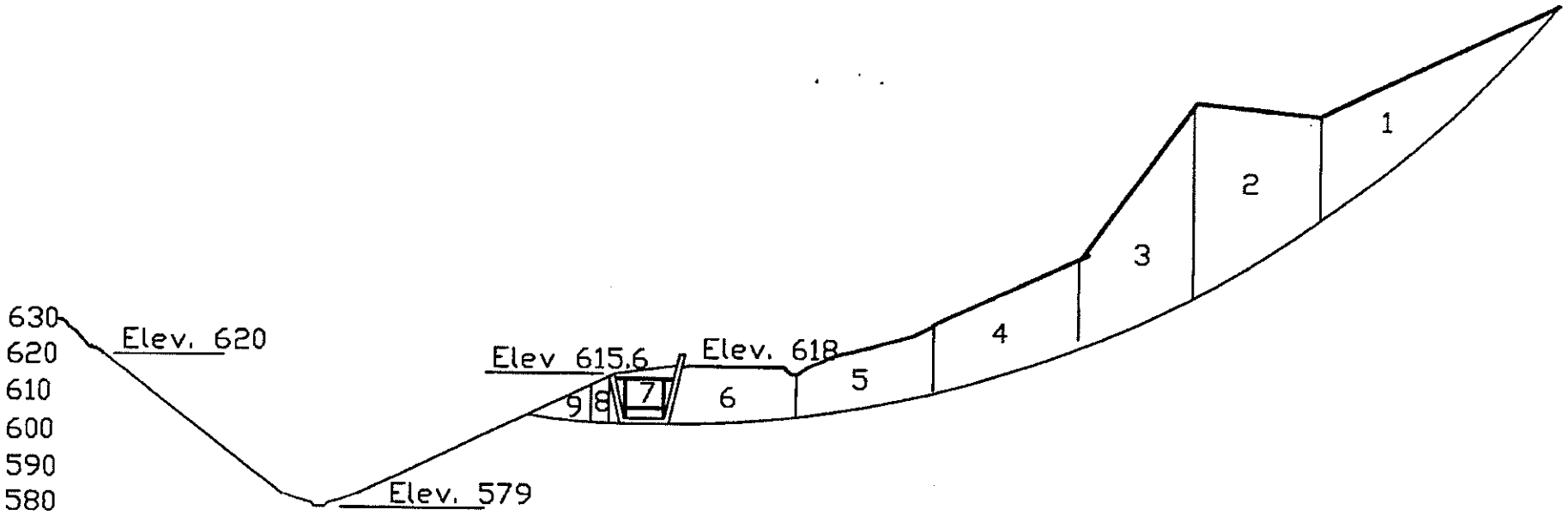


FIGURE A7 - CROSS SECTION H-H

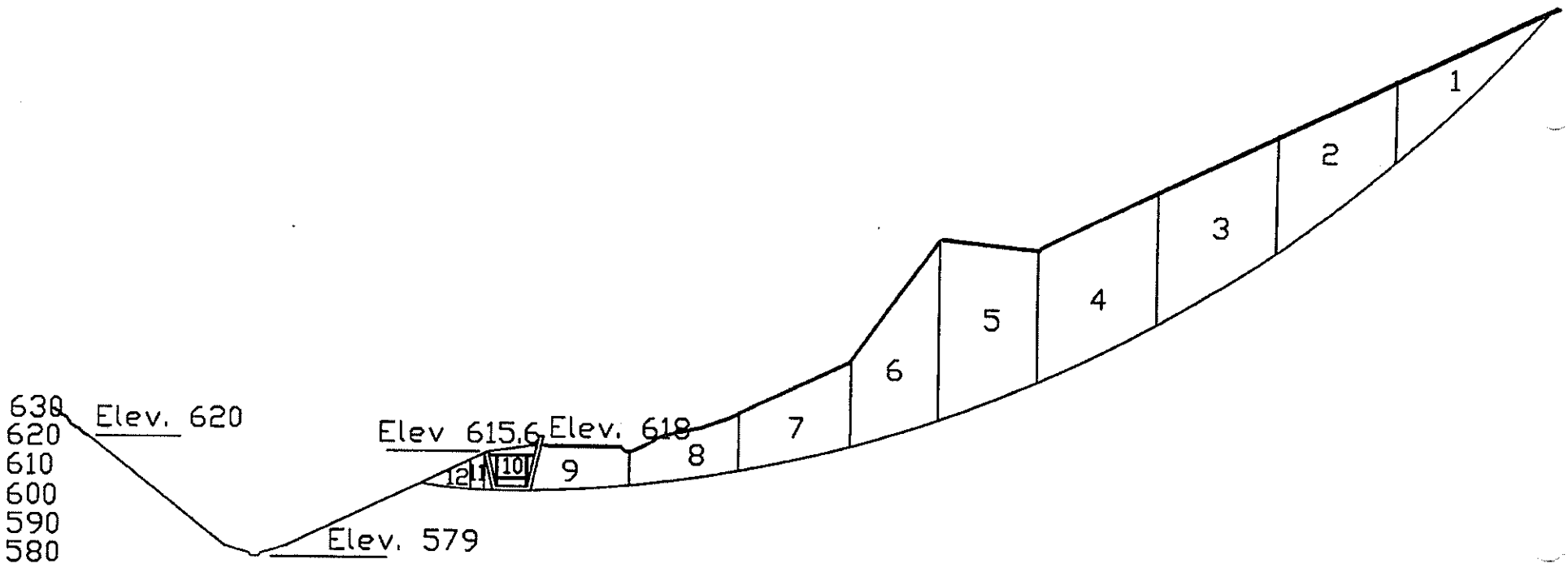


FIGURE A8 - CROSS SECTION H-H

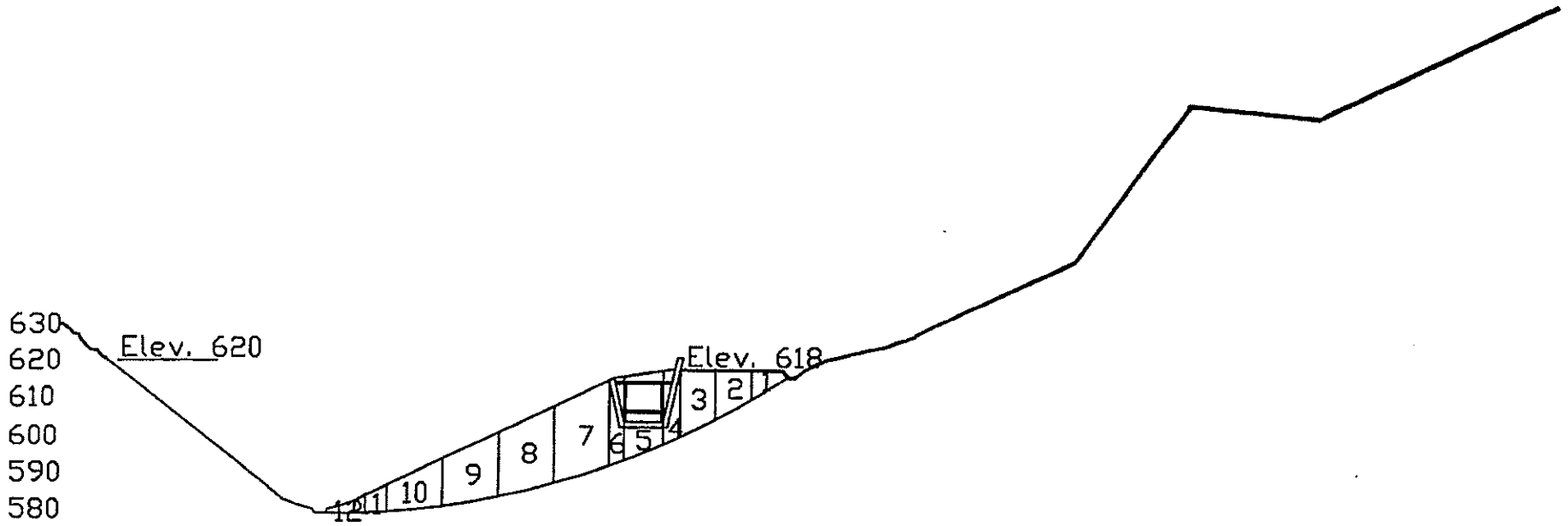


FIGURE A9 - CROSS SECTION H-H

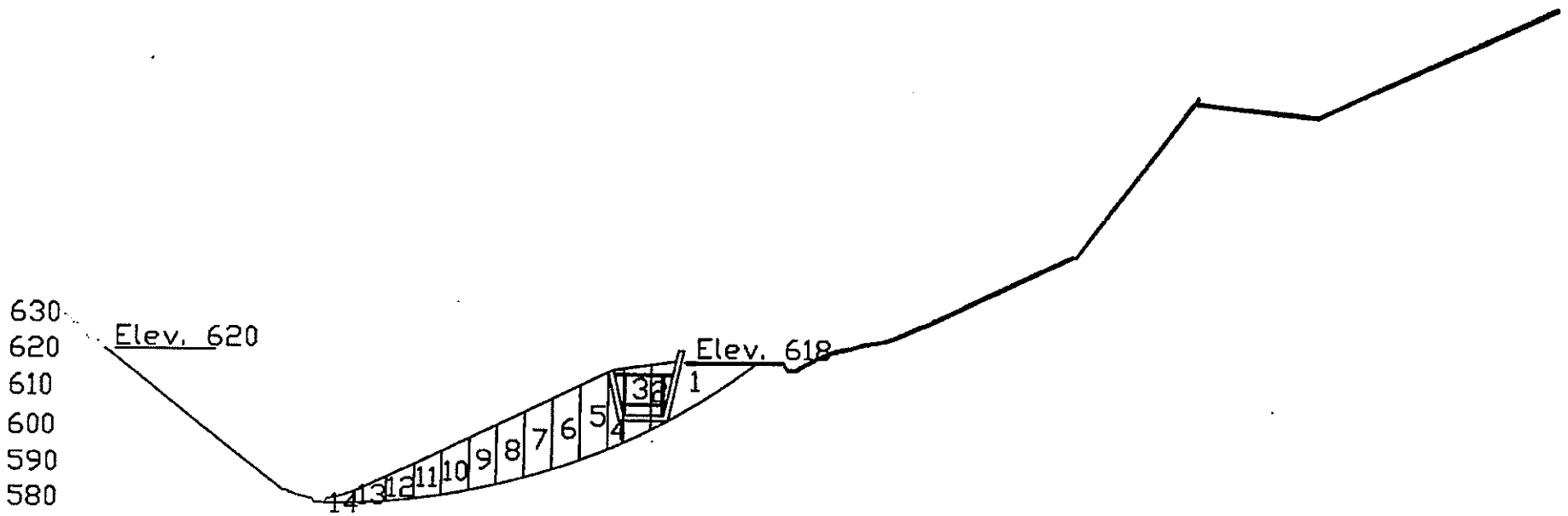


FIGURE A10 - CROSS SECTION H-H

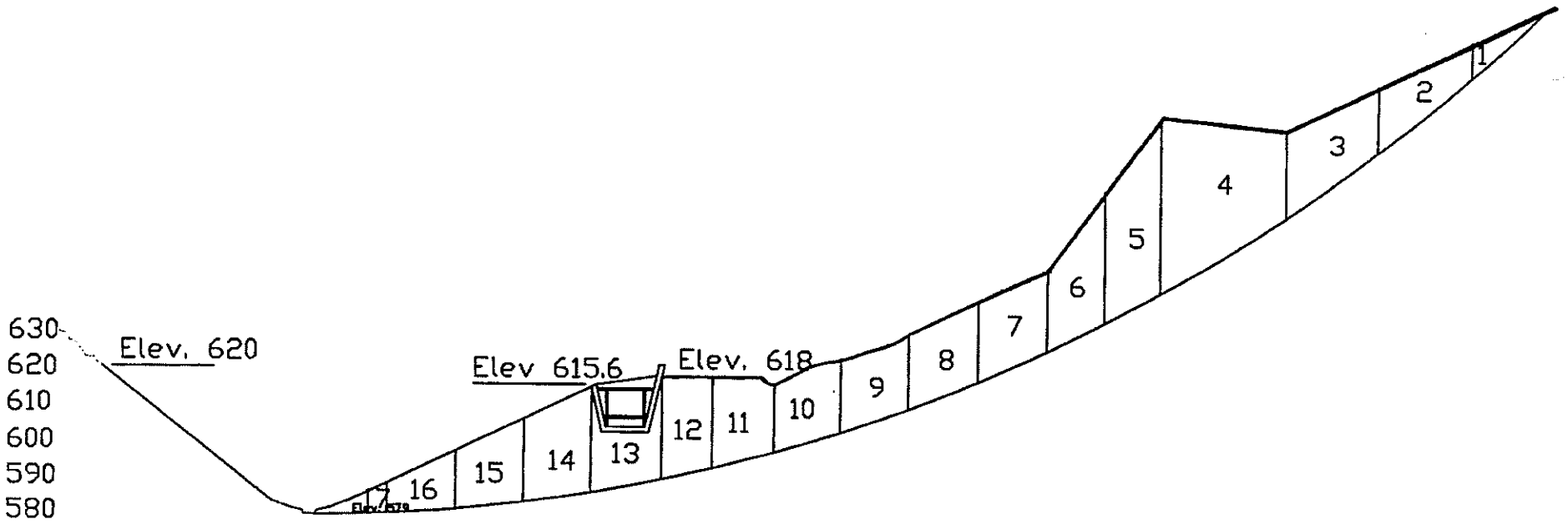
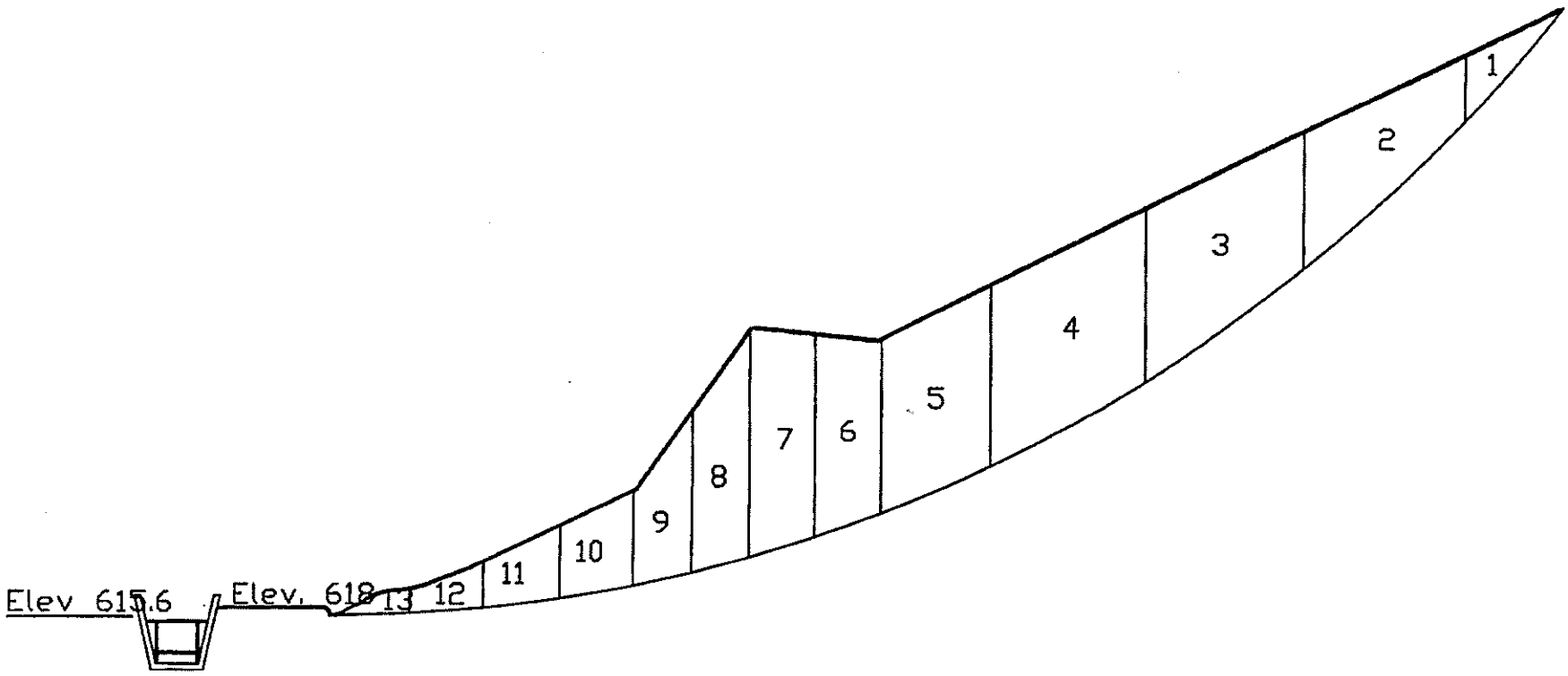


FIGURE A11 - CROSS SECTION H-H



Factor of safety: 1.0

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
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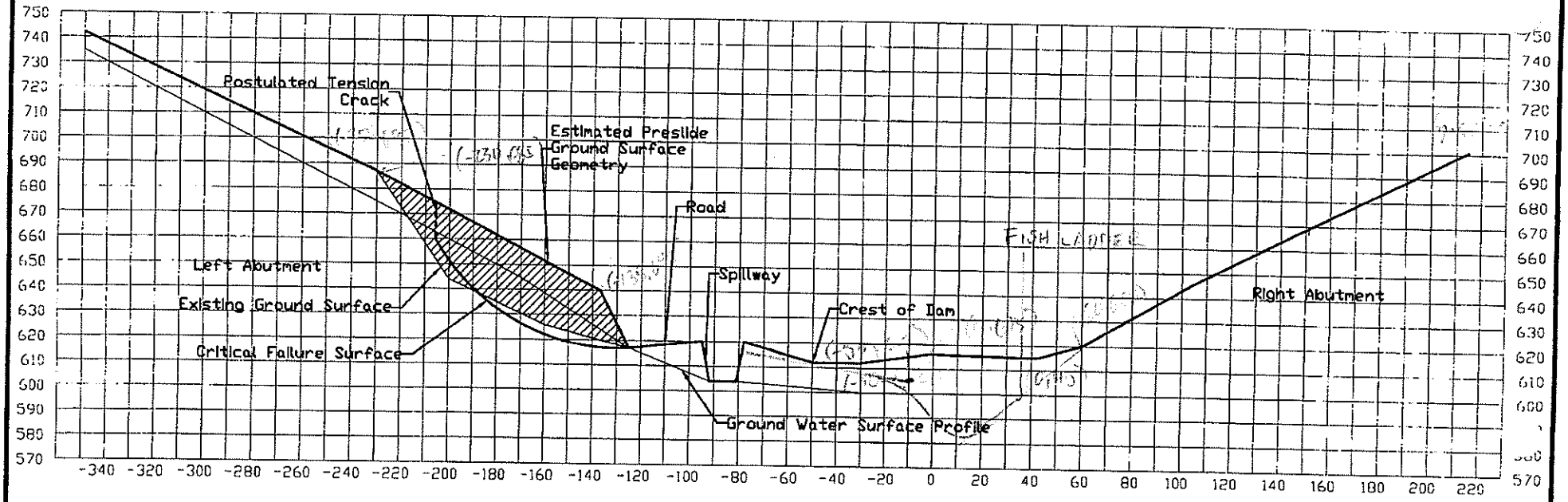

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT,
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

LOGGED BY:	York Creek Dam	California
DRAWN: VTran	Slope Stability	
CHECKED: —	Existing Slide	
DATE: 2/11/05	SCALE: 1:60	FIGURE: C-1

Factor of safety: 1.3

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
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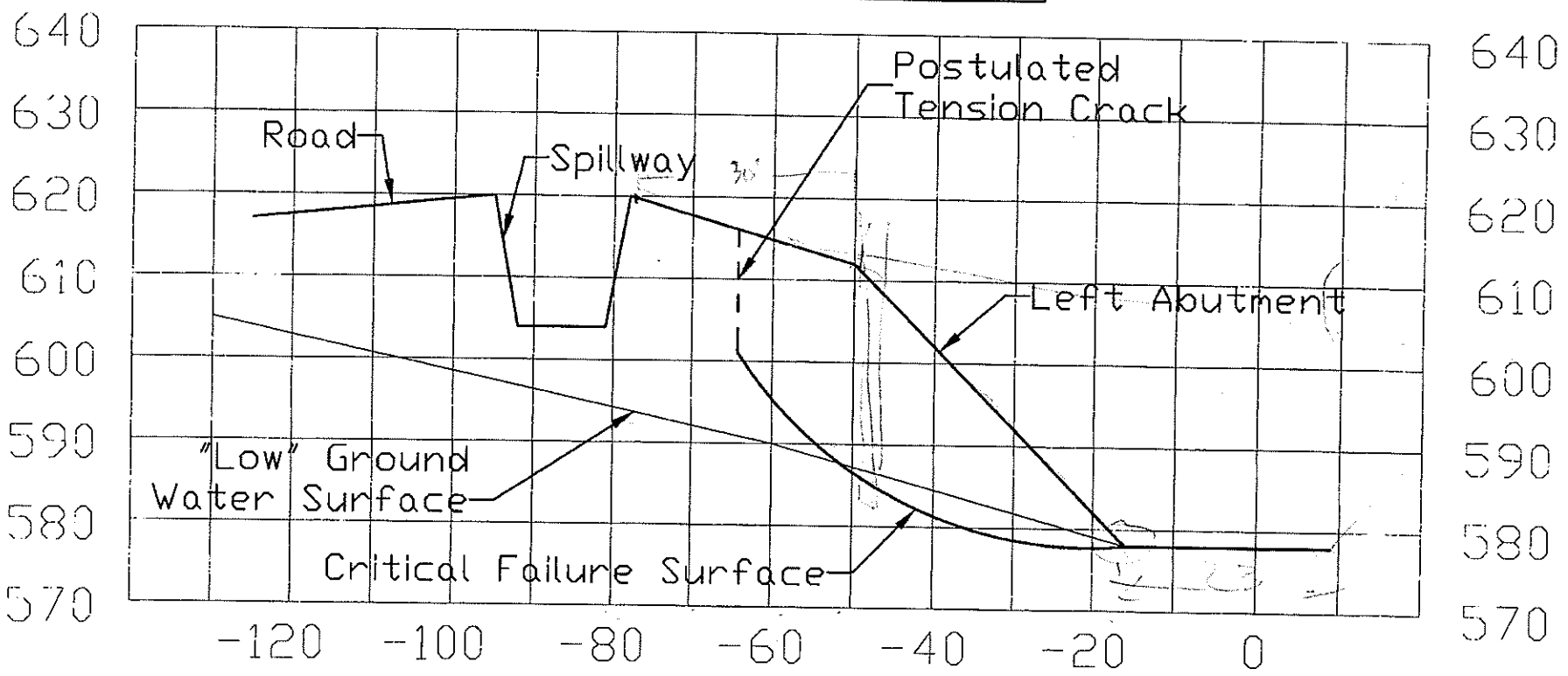

DEPARTMENT OF THE ARMY SACRAMENTO DISTRICT, CORPS OF ENGINEERS SACRAMENTO, CALIFORNIA		
LOGGED BY:	York Creek Dam	California
	Slope Stability	
DRAWN: VTran	Left Abutment Without Dam	
CHECKED: –		
DATE: 2/11/05	SCALE: 1:20	FIGURE: C-2

Factor of safety: 1.2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
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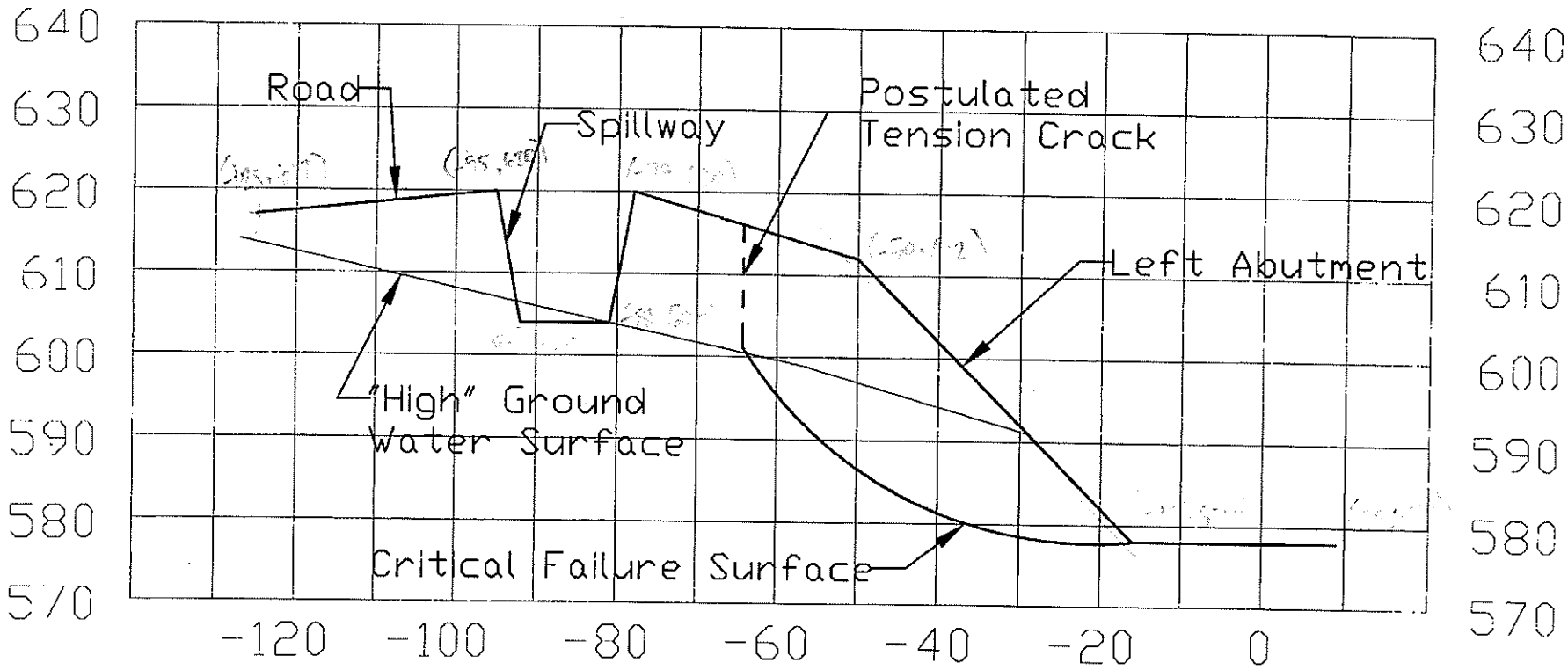
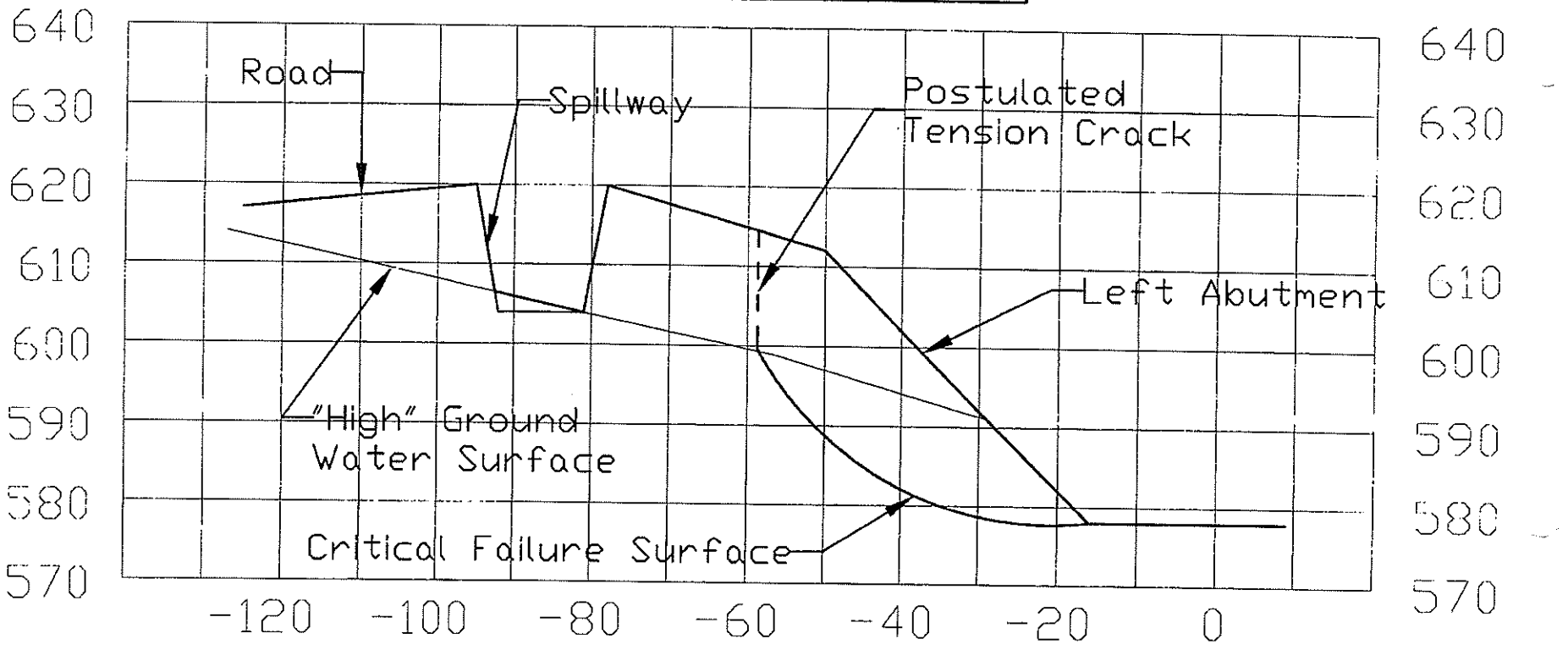


DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT,
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

LOGGED BY:	York Creek Dam	California
DRAWN: Vtran	Slope Stability	
CHECKED: —	Left Abutment Without Dam	
DATE: 2/11/05	SCALE: 1:20	FIGURE: C-3

Factor of safety: 2.2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Saturated Unit Weight	130	Cohesion 1000.0 Friction angle 37	Piezometric Line no. 1

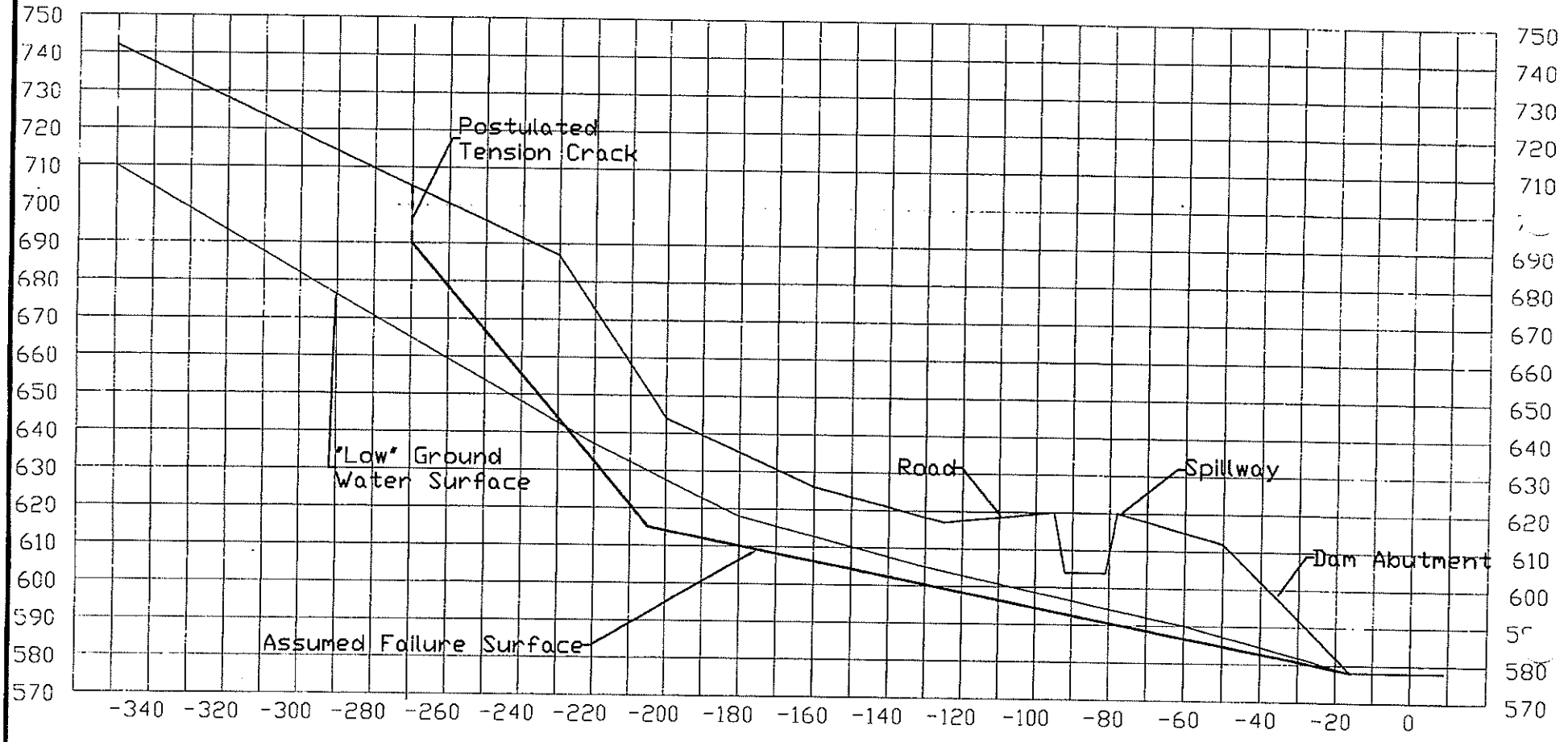


DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT,
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

LOGGED BY:	York Creek Dam	California
	Slope Stability	
DRAWN: Vtran	Left Abutment Without Dam	
CHECKED: —		
DATE: 2/11/05	SCALE: 1:20	FIGURE: C-4

Factor of safety: 1.2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Saturated Unit Weight	130	Cohesion: 600.0 Friction angle: 18	Piezometric Line no. 1



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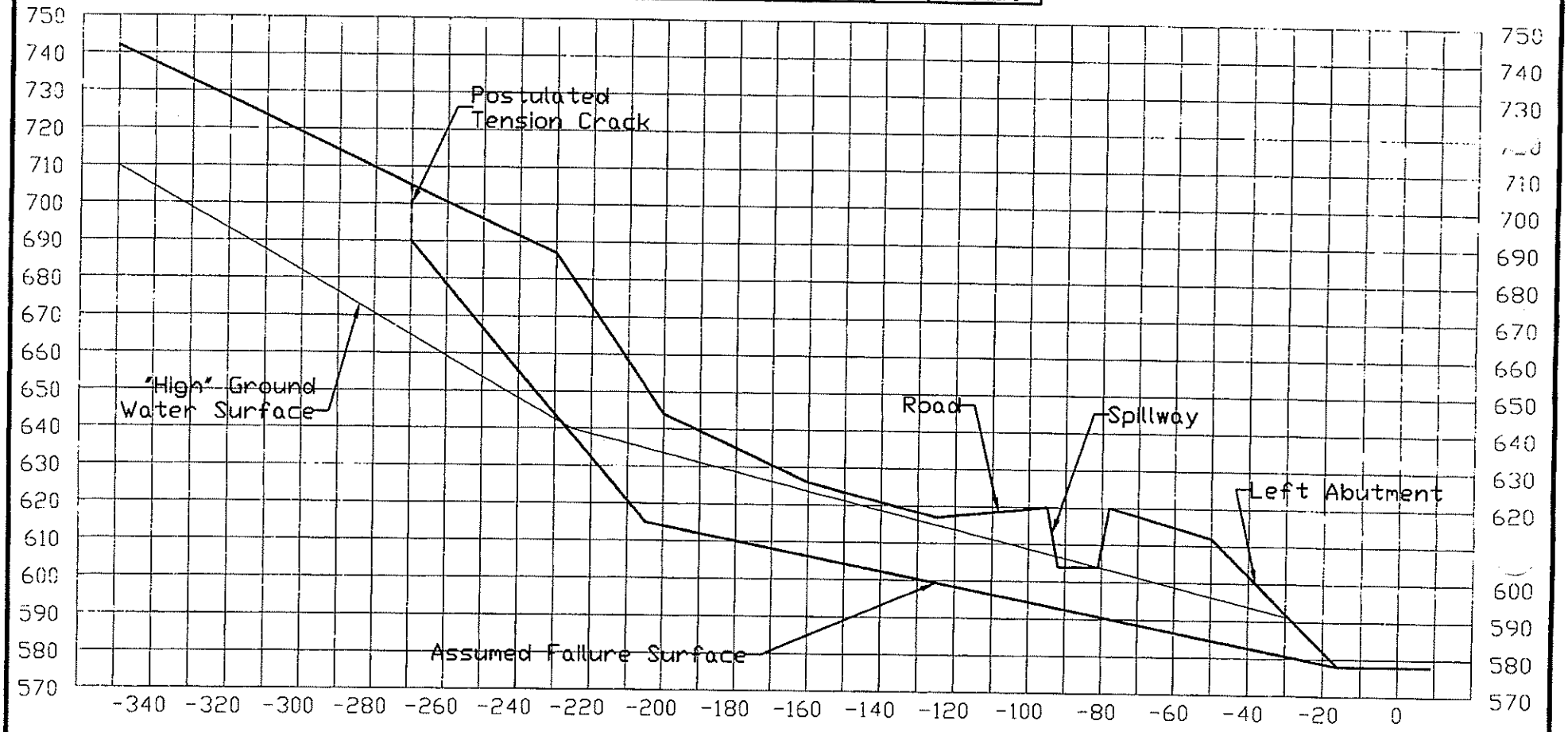
SACRAMENTO, CALIFORNIA

LOGGED BY:	York Creek Dam	California
DRAWN: Tran	Slope Stability	
CHECKED: —	Large Slide without Dam	
DATE: 2/11/05	SCALE: 1:40	FIGURE: C-5

Factor of safety: 1.1

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Saturated Unit Weight	130	Cohesion: 600.0 Friction angle: 18	Piezometric Line no. 1



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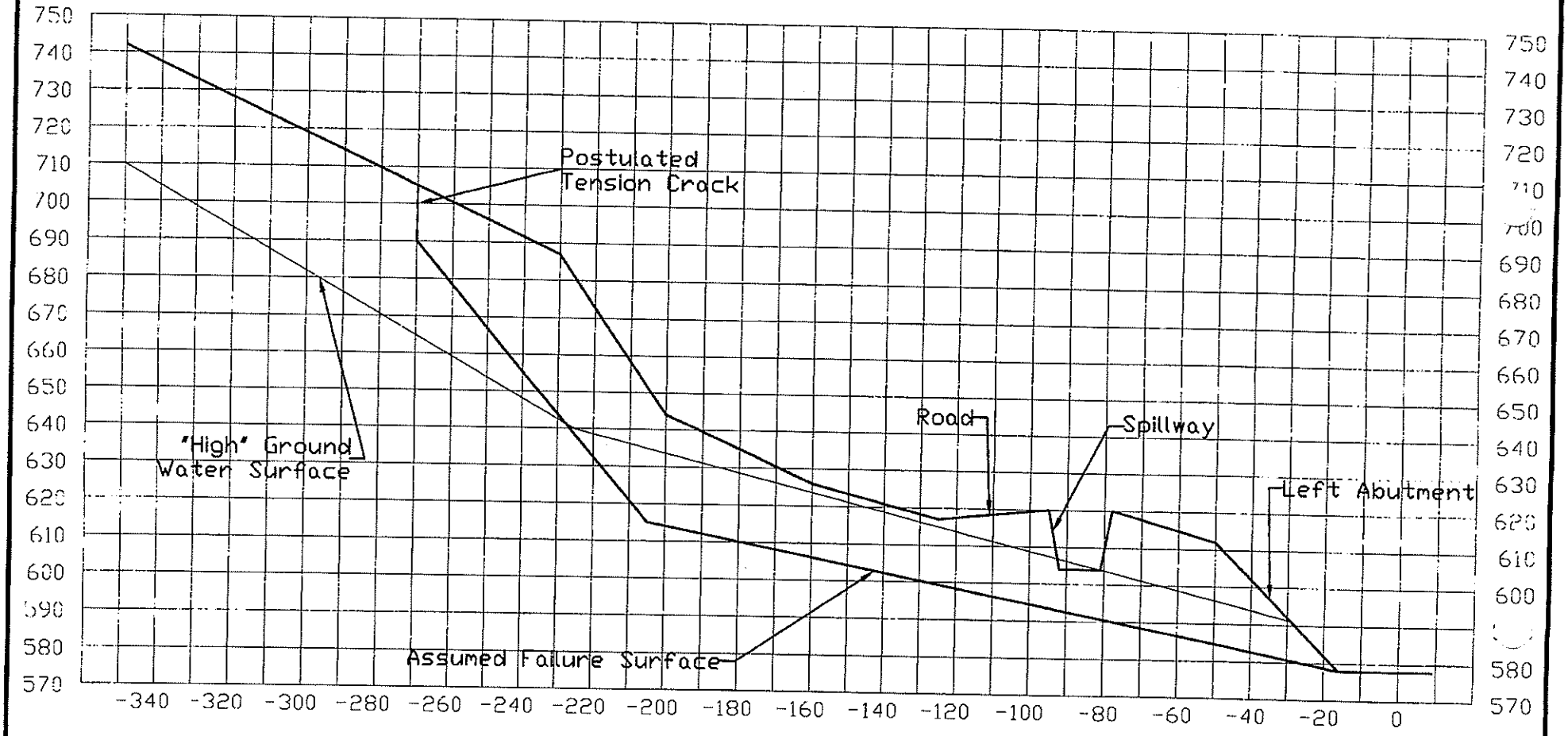
LOGGED BY:	York Creek Dam	California
DRAWN: VTran	Slope Stability	
CHECKED: —	Large Slide Without Dam	
DATE: 2/11/05	SCALE: 1:40	FIGURE: C-6

Factor of safety: 2.2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Saturated Unit Weight	130	Cohesion= 1000.0 Friction angle= 37	Piezometric Line no. 1



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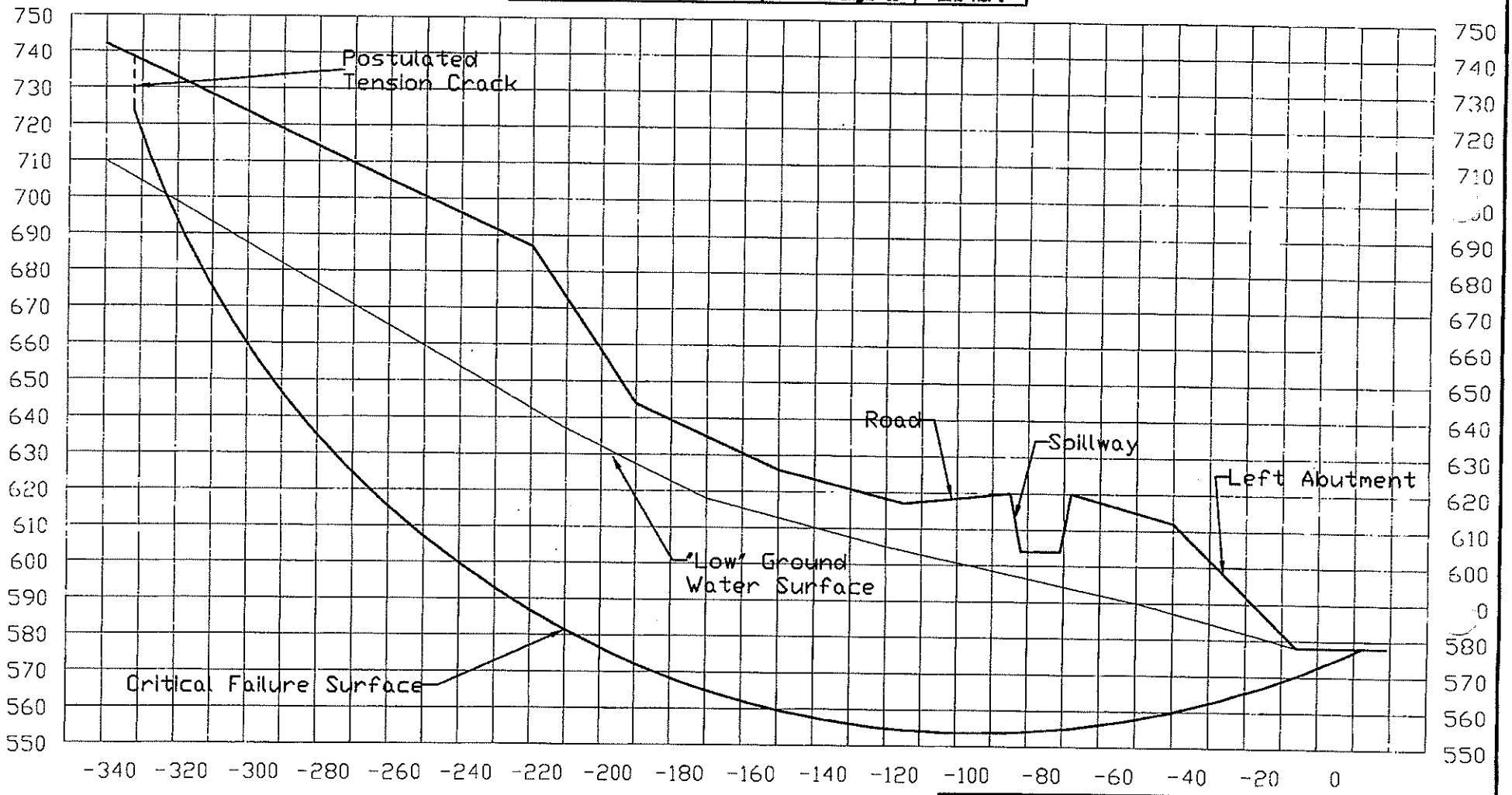
LOGGED BY:	York Creek Dam	California
DRAWN: VTran	Slope Stability	
CHECKED:	Large Slide Without Dam	
DATE: 2/11/05	SCALE: 1:40	FIGURE: C-7

Factor of safety: 0.9

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Saturated Unit Weight	130	Cohesion: 600.0 Friction angle: 18	Piezometric Line no. 1



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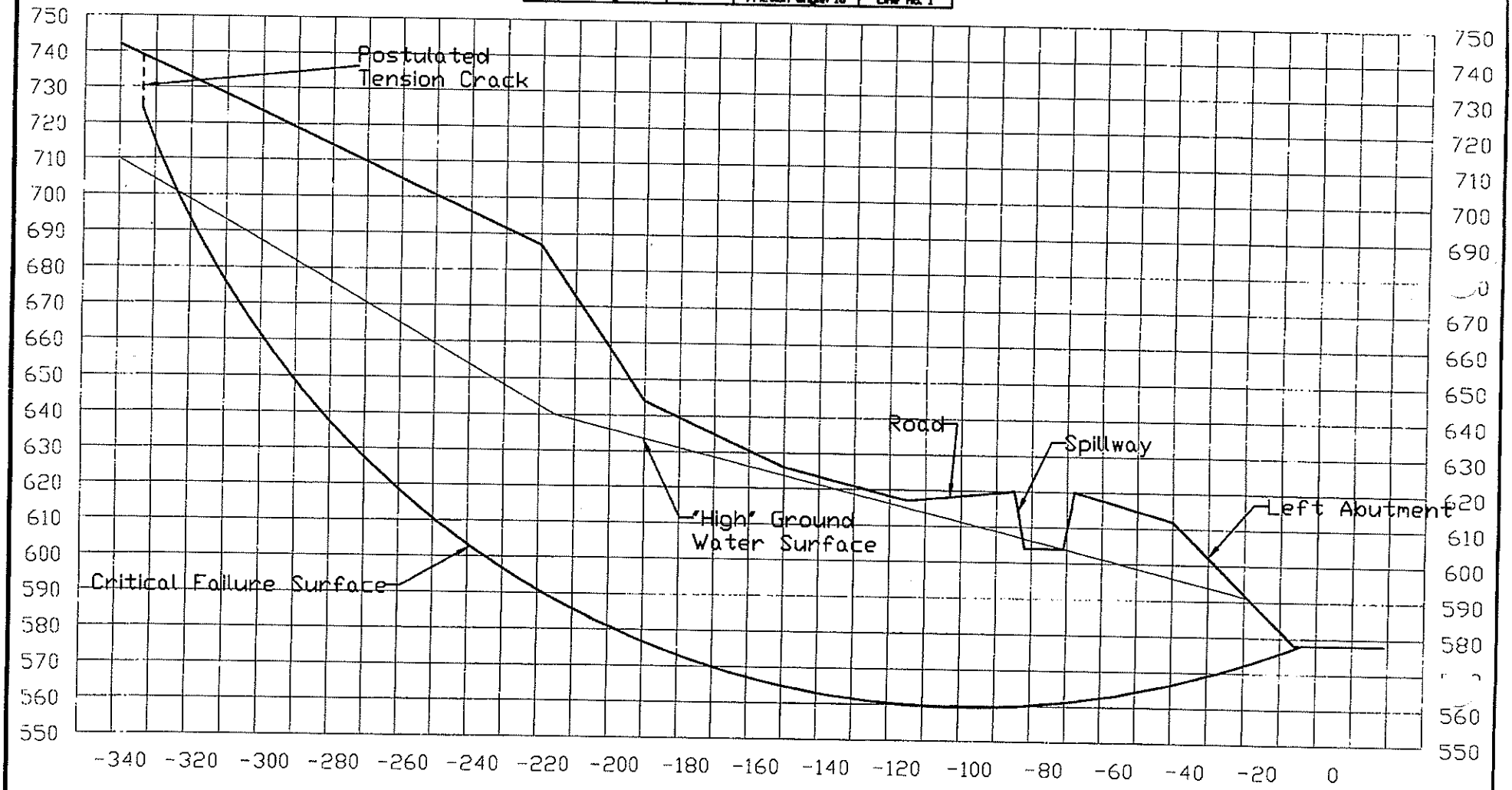
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

LOGGED BY:	York Creek Dam	California
DRAWN: VTran	Slope Stability	
CHECKED: —	Large Slide Without Dam	
DATE: 2/11/05	SCALE: 1:40	FIGURE: C-8

Factor of safety: 0.9

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Saturated Unit Weight	130	Cohesion 600.0 Friction angle 18	Piezometric Line no. 1



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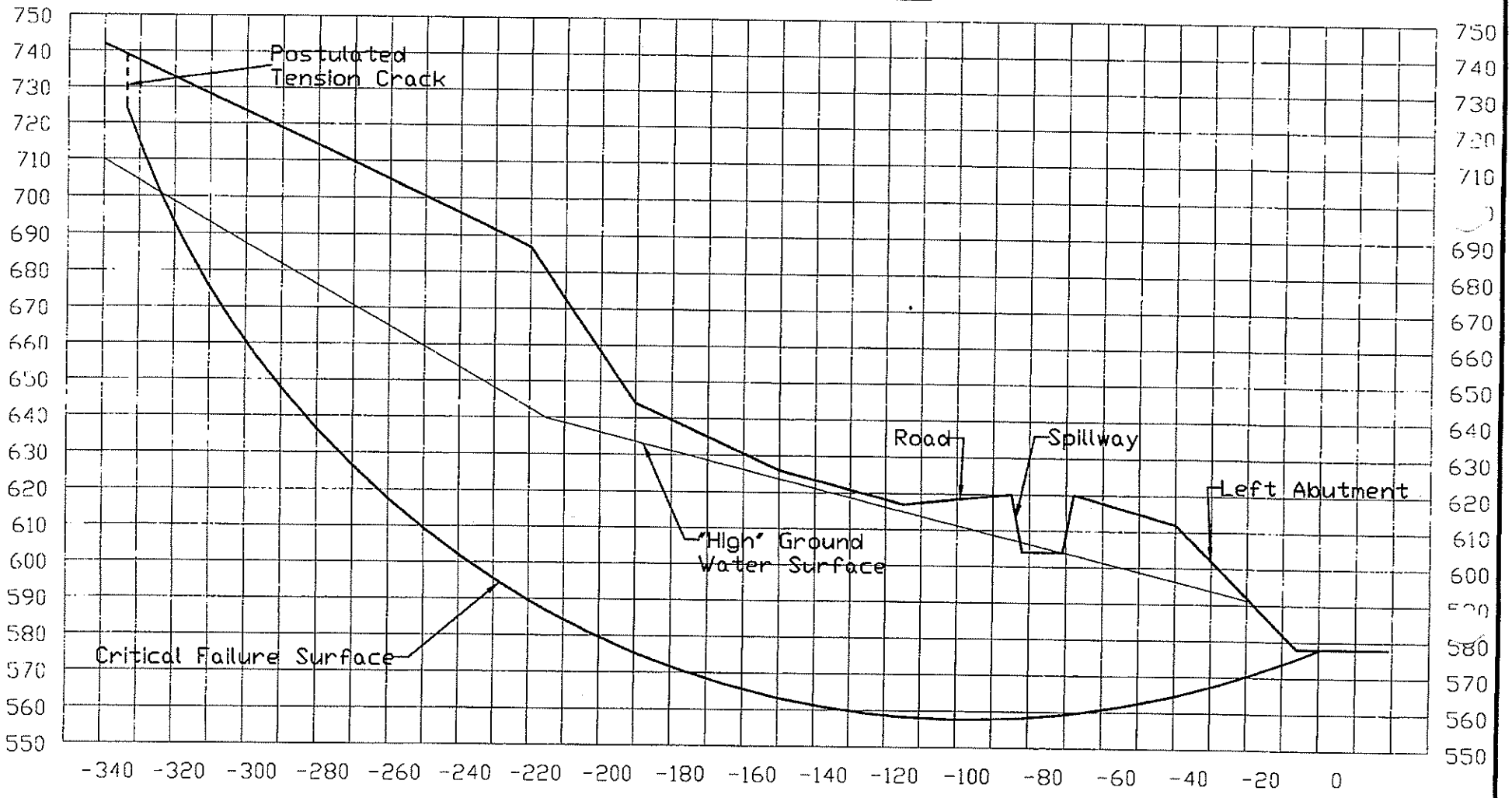
LOGGED BY:	York Creek Dam	California
DRAWN: VTran	Slope Stability	
CHECKED: —	Large Slide Without Dam	
DATE: 2/11/05	SCALE: 1:40	FIGURE: C-9

Factor of safety: 1.8

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Saturated Unit Weight	130	Cohesion 1000.0 Friction angle= 37	Piezometric Line no. 1



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SACRAMENTO, CALIFORNIA

LOGGED BY:	York Creek Dam	California
DRAWN: VTran	Slope Stability	
CHECKED: —	Large Slide Without Dam	
DATE: 2/11/05	SCALE: 1:40	FIGURE: C-10

1. INTRODUCTION

This report is a supplement to the Upper York Creek Dam Removal Project Site and Alternative Evaluation prepared by the U.S. Army Corp of Engineers, Sacramento District for the San Francisco District on 15 March 2005.

2. PURPOSE

The purpose of this report is to provide geotechnical recommendations for stabilizing an existing road if alternative 1 is selected – Removal of Entire Dam and Spillway.

3. BACKGROUND

Upper York Creek Dam is located on York Creek about 1.5 miles northwest of St. Helena, Ca along Spring Mountain Road. Upper York Creek dam is a 40 ft high earthen dam constructed in 1900. The spillway was later added to the left abutment in 1933. Routine maintenance was performed in the 1970s and in 1985 the dam was modified to bring it into compliance with safety standards. Upper York Creek Dam is owned and operated by the city of St. Helena.

Upper York Creek Dam Removal project is part of an ecosystem restoration project. The main objective is to reduce or remove barriers to migrating steelhead trout. There are currently 3 different alternatives being considered for fish passage and are as follows (see attached concept sketches).

- Alternative 1 includes removing the dam, spillways, outlet works, and all sediment behind the dam.

- Alternative 3 would remove the outlet works, a portion of the dam (notching), and most of the sediment behind the dam. A fish passage would be constructed using rock weirs at the original stream level.

- Alternative 4 would leave the dam relatively intact and would add a new fish passage structure either up and over the embankment, or through the existing left spillway.

Despite the purpose of this supplemental report, a variation on alternative 3 is the preferred geotechnical solution for both reducing barriers to migrating steelhead and maintaining a stable road. Locate the notch as far towards the right abutment (looking downstream) as possible. It is recommended that the spillway structure not be removed. Leave the spillway in place and backfill it to provide continued support for the existing road. Include a layer of drainage material and weep holes at the bottom of the spillway chute to allow for proper drainage.

4. RECOMMENDATIONS

There are several geotechnical structure options that could possibly provide adequate support to the road if alternative 1 is selected. Although alternative 1 provides the most complete removal of barriers to migrating steelhead, it requires the greatest effort to maintain the road. Extensive explorations, complex design, and large construction cost will be required compared to alternative 3 or 4.

- Option 1 is to have a single or double row of 3 or 4 foot diameter soldier piles spaced at approximately 4 feet on center to a depth of 100 feet adjacent to the road. Traffic control will be required. The spillway is to be removed after the piles have been placed. This option could provide sufficient protection against medium size slope instabilities of the road and the hillside adjacent to the road.

- Option 2 is to partially remove the existing spillway leaving the left spillway chute wall intact and installing one to three horizontal rows of tiebacks at 5 feet on center approximately 100 feet into native soil/rock. The diameter of the tiebacks could be as large as 12 inches thick. Additional reinforced concrete (approximately 12 in. thick) would also need to be added along the existing left spillway chute wall to increase its thickness. The tiebacks shall be installed before the spillway is partially removed. This option does not provide sufficient protection against medium to large size slope instabilities.

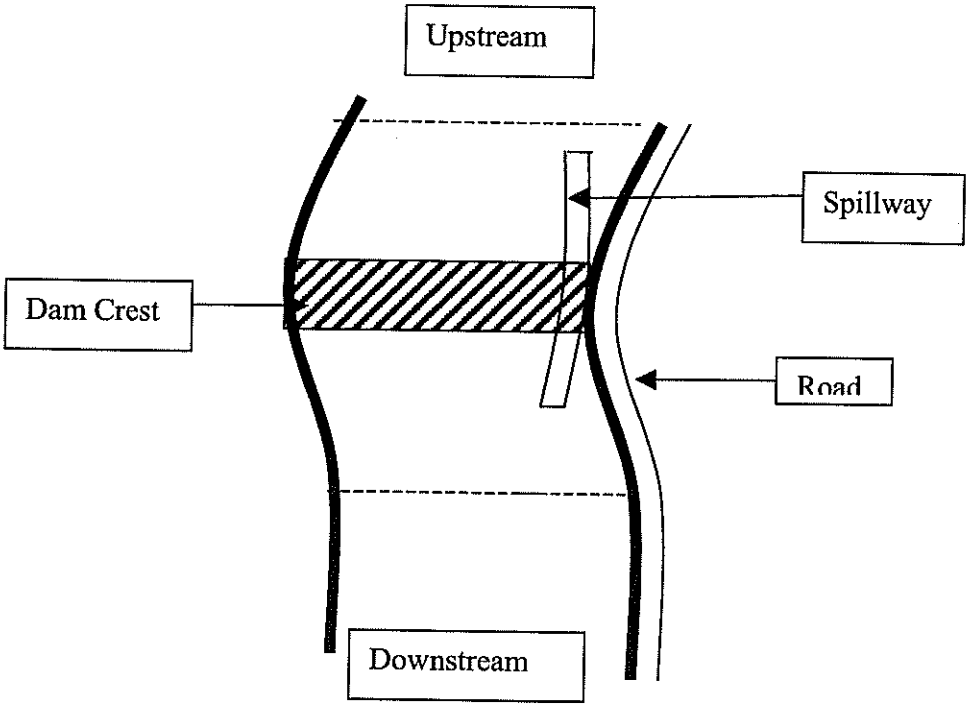
- Option 3 is to construct a continuous footing reinforced cantilever retaining wall. The approximate height of the retaining wall is 15 to 20 feet. Traffic control will be required. The retaining wall should be placed adjacent to the existing spillway chute. The spillway is to be removed after the retaining wall is constructed. Stabilizing the embankment cut while constructing the retaining wall may require additional equipment and construction efforts (i.e. shoring, soil nailing, and additional excavation). This option does not provide sufficient protection against medium to large size slope instabilities.

Further geotechnical investigation and analysis will be required when an alternative is selected. Additional geotechnical investigation may determine one or many of the options are not feasible. A combination of two or more options may be required to achieve adequate protection against slope instabilities. An in-depth subsurface exploratory program and investigation study will be needed to select the best construction option. These options are only to be considered if alternative 1 is selected as the preferred alternative.

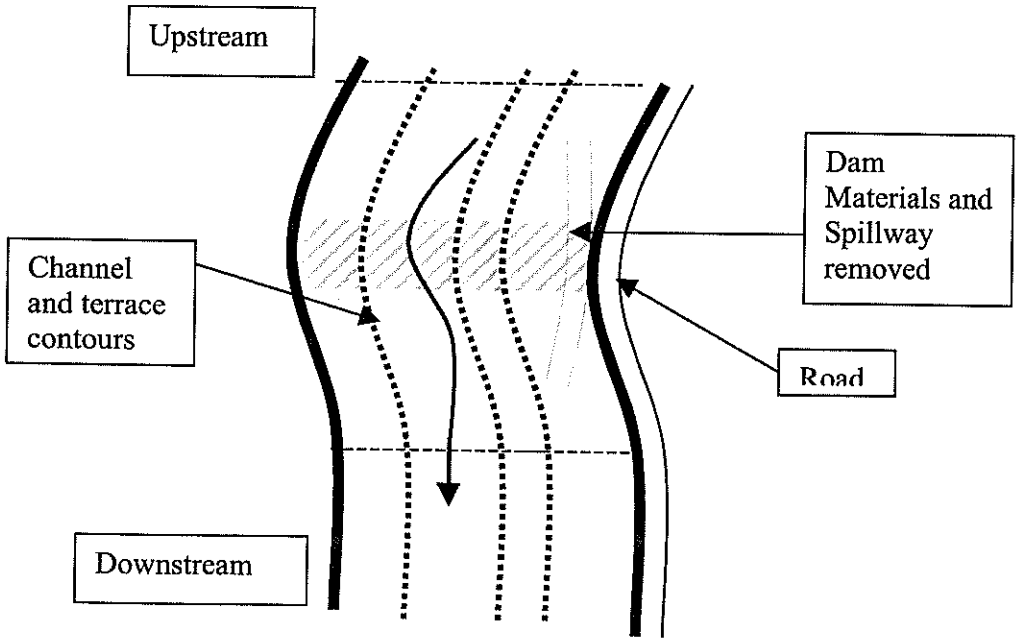
Vinh Tran
Dam Safety and Infrastructure
Support Section

Concept Sketches for Fish Passage Alternatives

Existing Condition

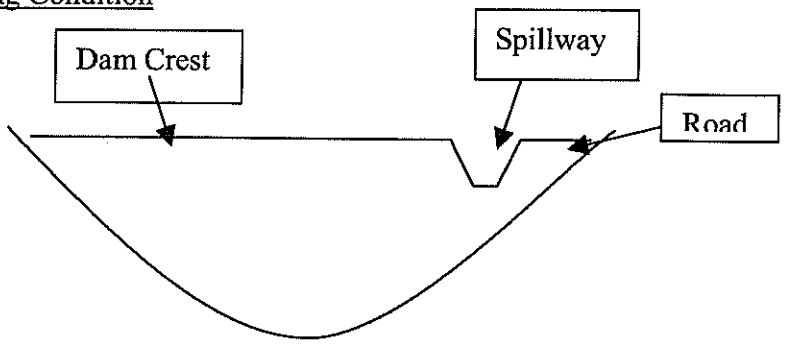


Alternative 1: Remove the Entire Dam and Spillway

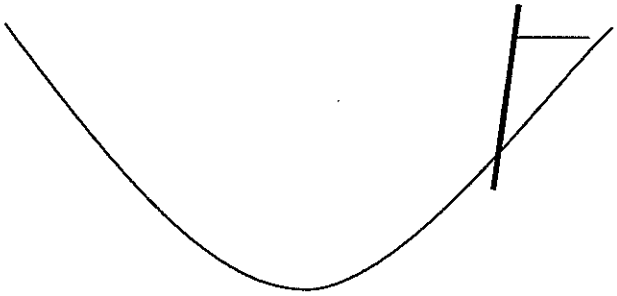


Dam Cross Section Sketches for Concept Fish Passage Alternatives (facing upstream)

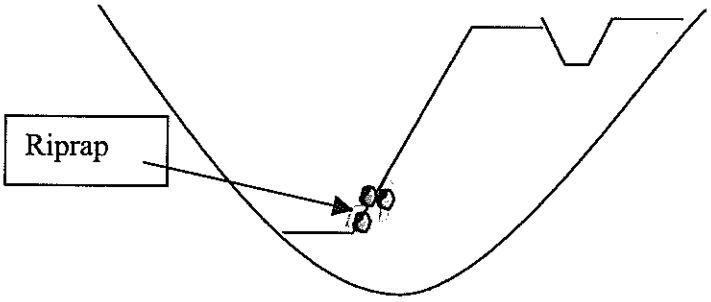
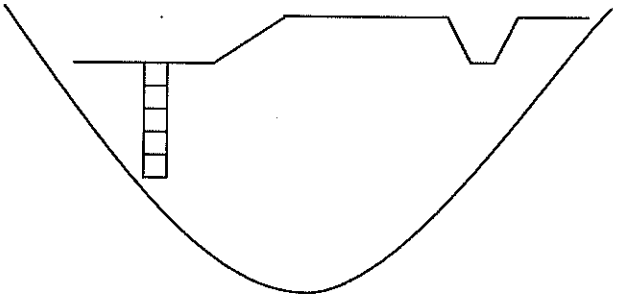
Existing Condition



Alternative 1: Remove the Entire Dam and Spill way



Alternative 3: Notch the Dam



Alternative 4 Fish Ladder and Notch